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THE UNIVERSITY OF ALBERTA

AN INVESTIGATION AND ANALYSIS OF A LANDSLIDE NEAR RAUSH VALLEY, BRITISH COLUMBIA

BY

MARK T. BRADSHAW

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE

OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

SPRING, 1988

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research for acceptance, a thesis entitled "*An Investigation and Analysis of a Landslide Near Raush Valley, British Columbia*" submitted by Mark T. Bradshaw in partial fulfillment of the requirements for the degree of Master of Science.

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Date: February 9, 1988

ABSTRACT

The CN Rail North Division line is plagued by many areas of slope instability on its passage through the Fraser River valley in central British Columbia. The surficial geology of the river valley is dominated by recent glacial lake clays of medium to high plasticity with discontinuous silt laminations. The presence of numerous alluvial fans and aprons submerged beneath the glaciolacustrine deposits results in widely fluctuating groundwater levels and, in some cases, artesian conditions.

The site, located at Mile 30.8 on the Tete Jaune Subdivision, has had a long history of slope instability dating from the early 1950's. Various costly maintenance techniques have been employed to combat these movements with little success.

Excessive slope movements triggered in 1983 by high rainfall and the improper placement of a stabilizing berm led to the initiation of major remedial works in 1984. These works included the installation of a counterfort drainage system and a stabilizing berm. Extensive instrumentation consisting of pneumatic piezometers, standpipes, and slope indicators were installed to verify design assumptions and determine the effectiveness of the remedial works.

This case history also provided an excellent opportunity to monitor the groundwater response to a number

of influences including surcharge loading during berm construction, installation of counterfort drainage, and seasonal rainfall and runoff fluctuations before and after drainage installation.

The results of the remedial works have been a cessation of track movements and a measurable reduction in groundwater level fluctuations caused by seasonable rainfall and runoff extremes.

A combination of field and laboratory testing, field porewater pressure and slope movement monitoring, and computer analysis of slope stability have been used to provide an integrated overview of a slope stability problem in a sensitive region.

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ABBREVIATIONS

Symbol	Description	Units
$\overline{B}$	pore pressure ratio ($\Delta u / \Delta \sigma_1$)	
C'	effective cohesion	kPa
c.m.p.	corrugated metal pipe	
F	factor of safety	
g.w.	groundwater	
H	height	m
I_p	plasticity index	%
K	coefficient of permeability	m/s
K_h	horizontal permeability	m/s
K_v	vertical permeability	m/s
kPa	kiloPascals	
kn	kiloNewtons	
m	meters	
mh	manhole	
s	seconds	
S	sensitivity	
SPT	standard penetration test	
t	time	
t_{50}	time to 50% equalization	s
t_{90}	time to 90% equalization	s
u	porewater pressure	kPa
w	water content	%

Symbol	Description	Units
w_l	liquid limit	%
w_p	plastic limit	%
w_n	natural water content	%
γ	unit weight	kn/m^3
Δ	change or increment	
μ	system flexibility	
σ'_n	effective normal stress	kPa
τ	shear stress	kPa
φ'	effective friction angle	degrees
φ'_r	residual strength friction angle	degrees

1.0 INTRODUCTION

The CN Rail North Division which branches from the line connecting Edmonton and Vancouver at Red Pass, travels in a northwesterly direction to Prince Rupert on the Pacific coast. Between Red Pass and Prince George the North Division follows the Fraser River in the physiographic region known as the Rocky Mountain Trench.

The Rocky Mountain Trench is a broad, flat bottomed valley in this area. The floor of the trench is dominated by recent glacial lake clays of medium to high plasticity with discontinuous silt laminations. These glacial clays are underlain in many areas by alluvial aprons and fans, particularly near the valley walls. Meltwater from the spring thaw is channelled through these interfingering alluvial deposits into the glacial lake clays, often causing artesian groundwater conditions.

Much of the valley floor is dominated by small farms and the meandering Fraser River. The many talus slopes and avalanche tracks visible on the steep valley walls give an indication of the location of the buried alluvial deposits.

At many locations along the highway roadsides and rail line cuts there is evidence of shallow rotational slope failures caused by seasonally high groundwater levels, overcutting or oversteepening of slopes, surcharging of embankment crests, and disturbance of the vegetation. The CN

Rail line is plagued by many such sites of slope instability on its passage through the Fraser Valley. Artesian conditions are often at the heart of the slope stability problem as the rail line follows the valley walls closely in some areas.

The Tete Jaune Subdivision Mile 30.8 site is located near Raush Valley, approximately 500 kilometers west of Edmonton as illustrated in Figure 1. The CN Rail line follows the Fraser River in the Rocky Mountain Trench which divides the Cariboo and the Rocky Mountains. The original rail line was constructed by the Grand Trunk Pacific Railway in 1914 and was later amalgamated along with several other smaller lines under the Canadian National Railways in 1923.

The original line was constructed using somewhat crude cut and fill techniques utilizing the local glacial lake clays with little or no foundation preparation. Very few records remain that document the maintenance procedures carried out on the rail lines. Maintenance generally consisted of frequent reballasting of the tracks to maintain grade. The available history ~~of~~ the sliding movements at the Mile 30.8 site dates from the early 1950's when frequent reballasting was necessary to counteract excessive track settlements.

Drainage measures were installed in the 1960's consisting of a drainage collector ditch upslope of the affected sliding mass and a series of culverts to channel water away from the area. This measure seems to have

improved matters until the mid-1970's when settlements increased and required repeated seasonal track liftings in the spring and fall, traditionally high rainfall and runoff periods. This period also saw the increase in axle loading and rail traffic which further aggravated conditions.

In 1981 a 20 metre wide granular berm was placed downslope of the tracks in an attempt to flatten the slope but this resulted in increased slope movement because of poor positioning.

The combination of increasing thicknesses of railroad ballast and the ill-advised berm placement combined to further surcharge the crest of the sliding mass. Railroad traffic was considerably hampered by the slope movements which resulted. The abnormally high rainfall of 1982 finally resulted in the formation of tension cracks and a small rotational slide in 1983. A visible headscarp and toe bulge were noted along with the formation of small springs at the toe of the slide.

The CN Rail construction staff performed a back-analysis of the 1983 slide using limiting equilibrium slope stability techniques to provide input parameters to the design of remedial works. The design of a full remedial works program was initiated to correct the problem and maintain railway traffic. The remedial works consisted of the installation of a counterfort drainage system and a stabilizing berm.

Extensive instrumentation consisting of a network of pneumatic piezometers, Casagrande-type standpipe piezometers, and slope indicators were installed to verify the design assumptions and determine the effectiveness of the remedial measures. The author became involved in the Mile 30.8 slope stabilization project during the design of the instrumentation system and assisted in the design, installation and monitoring of the instruments.

The objectives of this study were to further analyze the causes of the slope failure which took place at the Mile 30.8 site in 1983 and to evaluate the effectiveness of the remedial measures in halting the slope movements.

Additional limiting equilibrium slope stability analyses were performed to back-analyze the 1983 slope movements and investigate the influence of variations in the pore pressure conditions, sliding surface geometry, and the effect of the passing freight trains. A laboratory testing program was also undertaken to determine the shearing strength and index properties of undisturbed soil samples obtained at the site during the instrumentation program. Direct shear strength tests were conducted to obtain effective strength parameters for comparison with the results of the computer slope stability back-analyses and laboratory data from sites in the surrounding area.

The effectiveness of the remedial works in stopping further slope movements and controlling the ground water level fluctuations was evaluated through extensive

analysis of the field instrumentation data. The instrumentation also provided an excellent opportunity to monitor the groundwater response to seasonal rainfall and runoff fluctuations before and after remedial works construction.

Conclusions and recommendations are forwarded regarding the influence of the counterfort drainage system on the ground water conditions and the effectiveness of the remedial measures in stabilizing the slope.

2.0 SITE CONDITIONS

2.1 SITE DESCRIPTION

The Tete Jaune Subdivision Mile 30.8 site is located approximately 15 kilometres southeast of McBride, British Columbia in the Fraser River valley. The sliding area affects both the main line and the Raush Valley siding and extends approximately 300 metres in length. The CN line is located on the south side of the Fraser River and follows the river at a distance of approximately 500 meters. Figures 2 and 3 illustrate a plan view and typical slope profile at the site.

The railway tracks follow the crest of a slope dipping northeast to the river at a gradient of approximately 5 horizontal : 1 vertical. Upslope and downslope of the tracks the land flattens into meadowland with some areas which become seasonally marshy. A parallel access road used by local residents is benched into the hillside approximately 50 metres downslope of the tracks.

A large drainage ditch upslope of the tracks is positioned to intercept runoff and channel it away from the sliding area. A similar ditch is located on the upslope side of the access roadway. Standing water is often found in the ditch adjacent to the rail line during high rainfall periods.

Much of the land in the flat bottomed river valley has been cleared and is used for small farms. All trees

have been removed from the area experiencing sliding instability to permit construction of remedial measures. Many small alluvial fan and apron deposits in the area have been exploited by local farmers and contractors as borrow material for roadways.

Oversteepening of roadway cuts and embankments in the soft glacial lake clay soils has led to the development of many small rotational slumps along the roadsides similar to those occurring along the rail lines.

2.2 REGIONAL GEOLOGY

The Tete Jaune Subdivision Mile 30.8 site is located in the physiographic region known as the Rocky Mountain Trench in eastern British Columbia as illustrated in Figure 4. The Rocky Mountain Trench is a flat-floored, steep-walled valley, bounded by the Rocky Mountains on the east and the northern range of the Columbia Mountains on the west (Campbell et al, 1972).

The trench is a structurally controlled erosional feature that extends almost 1500 kilometres in length in a northwesterly/southeasterly direction. Four major river systems dominate the floor of the trench at various locations: the Kootenay, Columbia, Canoe and Fraser Rivers, with low divides separating their headwaters (Holland,1964).

The structure of the trench varies considerably over its length. Although no single theory explains its

origin, it is generally believed that the erosional formation originated early in the Tertiary and was completed by the end of the Pliocene. The western boundary of the trench is formed by the Cariboo Mountains, the northernmost part of the Columbia Mountain Range, and the eastern boundary is formed by the Rocky Mountain Park Ranges.

The glacial history of the area is well documented by Tipper (1971). Tipper identifies an extensive glacial period in the early Pleistocene when ice accumulated in a thick central dome and later flowed radially outward to cover the Coastal, Cariboo and Rocky Mountain ranges. Little evidence is left of this early continental glacial period as the subsequent Fraser glaciation eliminated most earlier glacial features. Figure 5 illustrates the direction of ice movements in central British Columbia.

The Fraser ice sheet originated with the formation of alpine glaciers in the higher mountain areas, principally the Coastal and Cariboo Mountains. These alpine glaciers grew to form coalescent valley glaciers which overrode much of the topography and flowed into the low-lying areas.

Northeasterly moving ice from the Coastal Mountains met southwesterly moving Cariboo mountain ice resulting in a deflection of ice flow as illustrated in Figure 6. Two ice masses formed, one moving in a southeasterly direction and the other flowing northeasterly towards the Rocky Mountains. The northeasterly flowing mass was not of sufficient size to override the Rocky Mountains

and was diverted southeasterly up the Fraser River Valley and northerly down the Parsnip River Valley.

During deglaciation several broad ice masses were successively stranded by the retreating ice. These masses downwasted independently, leading to the formation of large glacial lakes confined in the relatively narrow valleys. Glacial Lake Prince George formed as ice retreated down the Fraser River Valley. It is believed that the lake level was maintained about elevation 760 metres resulting in the deposit of thick sequences of lacustrine sediment below this elevation.

Later readvances of the ice sheets originated again from the Cariboo Mountains and flowed to the southwest. Figure 7 illustrates the extent of this limited ice readvance. The Fraser Valley was likely dammed again producing further deposition of lacustrine sediments. Glacial Lake Prince George is believed to have extended as far down the Fraser River Valley as Tete Jaune Cache, accounting for the presence of glaciolacustrine sediments on the valley floor.

2.3 SITE STRATIGRAPHY

Site investigation and instrumentation programs were conducted in 1984 and 1985 by CN Rail to gain site specific geotechnical information relating to the engineering properties of the underlying soils and the groundwater conditions present at the site. The 1984 program consisted of 18 auger holes for the installation of slope indicators and Casagrande-type standpipe piezometers. The 1985 program, in which the author participated, consisted of 63 auger holes for the installation of pneumatic piezometers and additional slope indicators.

A typical geologic cross-section developed from borehole data obtained during the 1984 and 1985 drilling programs is presented in Figure 8. The surficial unit consists of 1-2 metres of dessicated, dry, silty clay containing many organic rootlets. Underlying the dessicated cap is 4-5 metres of brown medium to high plasticity silty clay that is uniform, moist, and contains discontinuous silt lenses. The silt lenses vary in thickness from 1 mm to 3 cm and have a silvery colour indicating their high mica content. No continuity of lensing was found from borehole to borehole on a 20 metre grid spacing.

Underlying the brown clay is a grey-brown to dark grey silty clay unit similar in structure to the overlying unit but sometimes exhibiting a slight blocky or nuggetty structure with lower moisture content indicating that this

clay may have originated from an earlier glacial advance and may have been exposed to freezing and/or drying cycles.

The interface between the two clay units lies approximately parallel to the original ground surface. When this information is coupled with the noted inclination of the silt lenses, it appears that these sediments were deposited on the sloping margins of a glacial lake, with bedding dipping towards the centre of the valley.

Previous investigations conducted by CN Rail at other sites in the area often identified thick underlying sequences of dense sand and gravel, likely representing alluvial fans and aprons buried by the lake sediments. A small fan deposit exists approximately 500 metres upslope of the Mile 30.8 site at the break in slope on the intersection of the valley floor and the steep valley wall. The angularity of the granular material and the inclusion of large angular bedrock pieces suggests that this deposit is part of a talus slope that may underlie the site.

Upslope of the main tracks the land is flat-lying with water ponding in low-lying areas. The water table is very near the ground surface throughout this area.

No artesian porewater conditions were discovered on the site itself, with the water table generally lying 2-3 metres below the ground surface.

Air photograph interpretation and use of previous studies produced for CN Rail have not indicated the presence of any previous or remnant slides in the immediate area

surrounding the Mile 30.8 Site. Some remnant slides have been identified, however, within a few kilometres of the site.

2.4 ENGINEERING MATERIAL PROPERTIES

A wealth of geotechnical information was available from CN Rail Engineering relating to the engineering material properties of the glaciolacustrine clays typical of those dominating the Mile 30.8 Site. The widespread occurrence of slope movements in the Fraser Valley area has required many field investigations by CN Rail geotechnical crews and independent consultants. This information was supplemented with a small but specific program of field and laboratory tests conducted by the author.

Previous investigations at sites located within 20 kilometres of the Mile 30.8 Tete Jaune subdivision have yielded valuable information including laboratory index testing, downhole vane and laboratory shear strength determination, air photograph geological interpretation, groundwater data, location of remnant slides and back analysis of previous slope failures.

This substantial body of information is supplemented by site specific data obtained during the Mile 30.8 site investigations and subsequent laboratory testing. Observations regarding the regional geology and site-specific observations made during the instrumentation and

remedial works construction phases help to give a clear picture of the engineering properties of the lacustrine clays present at the site.

VISUAL INSPECTION

A visual inspection of the soil samples recovered during the drilling program indicates that the clay has no visible laminations and is massive and uniform in structure. Some samples extracted from the split spoon sampler exhibited a slightly blocky structure with occasional indications of drying episodes. Silt nodules were present throughout the deposit ranging in size from 1-10 mm.

The silt lenses frequently encountered were often inclined at an angle of 70-85° to the core axis and varied in thickness from a few millimeters to 4 cm. These lenses exhibit little cohesion and are shiny silver in colour. Individual mica flecks which can be discerned upon inspection give this material a slippery feel. Lensing is discontinuous over a borehole spacing of 20 metres.

Although most boreholes remained dry during drilling, there were a few exceptions. A few boreholes remained dry until a silt lens was encountered and then immediately flooded. It is apparent that some of these lenses are water-bearing and could contribute a secondary permeability to the formation.

The ground surface has been reworked at various times during track maintenance and during the placement of the small granular berm in 1981. Varying thicknesses of gravel and sand were encountered at the top of many of the boreholes .

INDEX TESTING

Pocket penetrometer and Torvane tests were performed on soil samples immediately after recovery. Typical undrained strengths were measured at 40 to 125 kPa for the Torvane and 35 to 135 kPa for the pocket penetrometer.

During the 1984 drilling program, continuous undrained shear strength profiles were obtained by CN Rail geotechnical crews using a Nilcon downhole vane. These profiles consist of peak and remoulded strength determinations. Some profiles indicate the presence of a stiff unit at the ground surface likely representing the dessicated clay cap. Shear strengths at depth vary widely with no discernible pattern . Peak undrained strengths vary between 75 and 150 kPa and remoulded strengths vary between 10 kPa and 50 kPa. Remoulded undrained strengths are uniform with depth. The sensitivity of these deposits varies between 4 and 8. No clear pattern emerges from the in situ vane results that differentiates the upper and lower

clay units. This in itself indicates the boundary may be primarily a colour contrast.

Moisture content and Atterberg Limits were measured on some borehole samples. Moisture contents were generally high ranging between 30% and 55%. Again no clear difference between stratigraphic units was evident.

Atterberg Limit tests were performed to yield a continuous profile in one borehole. Figure 9 illustrates the data obtained from the Mile 30.8 site plotted against CN data obtained at other sites in the Fraser Valley. The plasticity chart classes all samples as high plasticity inorganic clay. Although some CN data at nearby sites indicates the presence of a high plasticity unit overlying a medium plasticity grey-brown silty clay, no contrast was noted in the soil plasticity at the Mile 30.8 site.

From the index testing data there is no clear indication of a contrast between the upper and lower clay units on the basis of undrained strength, moisture content or plasticity. These units are likely very similar in composition and stress history, and for practical purposes can be considered one unit. On the basis of the index and classification testing the lacustrine clay would be described as a medium to stiff, high plasticity silty clay. The Unified Soil Classification is CH.

LABORATORY SHEAR STRENGTH TESTING

A series of multi-stage direct shear strength tests were performed in the University of Alberta soils laboratory to determine the residual strength of the high plasticity silty clay and micaceous silt lenses. Samples were selected from undisturbed Shelby tube samples recovered during the site investigation and instrumentation program conducted at the Mile 30.8 site in the spring of 1985.

The purpose of this limited series of tests was to determine the shearing strength parameters which could be expected to be mobilized at the time of the 1983 rotational slide. The documented history of the slope movements taking place over the 70 year history of the line are believed to have been sufficient to reduce the operative strength of the lacustrine deposits to near-residual values.

Three samples were selected for the direct shear tests. These samples were obtained from Shelby tube samples extracted from depths of 6.7 to 8.0 metres below the ground surface. This depth corresponds to the depth of the 1983 sliding surface as inferred from slope indicator profiles. All samples were consolidated in the shearbox prior to shearing and sheared over a range of normal stresses consistent with the original in situ stress conditions.

Figure 10 illustrates the resulting residual strength envelopes developed for the three samples tested. Test #1 corresponds to the testing of a highly micaceous

silt lens while Tests #2 and #3 were performed on samples of high plasticity silty clay. The resulting residual strength parameters varied between $\phi' = 7^{o}$ to 12^{o} with C' values assumed to be zero.

This data is tabulated in Table 2 along with residual and peak shear strength data obtained from other CN Rail sites in the Tete Jaune Subdivision.

In situ falling-head permeability tests were conducted in the Casagrande-type standpipe piezometers in August 1986, approximately two years after their installation. Interpretation of the test data yielded estimates of the range of the in situ coefficient of permeability . The test procedure and interpretation of the field results are discussed in detail in Section 4.4.

TABLE 1

SUMMARY OF MATERIAL PROPERTIES
MILE 30.8 GLACIOLACUSTRINE CLAYS

ϕ_r' = 7^o - 12^o (direct shear tests)

C' = 0 (assumed)

w_n = 30% - 55% (average = 40%)

w_l = 68% - 97%

w_p = 27% - 35%

I_p = 39% - 63%

Sensitivity = 4 - 8

C_u = 75 - 150 kPa (in situ Nilcon Vane)

C_u = 35 - 135 kPa (pocket penetrometer)

C_u = 40 - 125 kPa (Torvane)

C_u (remoulded) = 10 - 50 kPa (in situ Nilcon Vane)

K = 1×10^{-10} - 3×10^{-8} m/s (determined from in situ falling-head permeability tests)

TABLE 2

SUMMARY OF SHEAR STRENGTH PROPERTIES MEASURED AT THE PROJECT SITE AND OTHER NEARBY SITES IN THE FRASER VALLEY

SITE MILEAGE	TYPE OF SOIL TESTED	PEAK STRENGTH PARAMETERS	RESIDUAL STRENGTH PARAMETERS
Tete Jaune Subdivision			
30.8	grey-brown silty clay (CH)	------- -------	C'= 0 φ_r'= 12°
30.8	grey silty clay (CH)	------- -------	C'= 0 φ_r'= 7°
30.8	micaceous silt lens (ML)	------- -------	C'= 0 φ_r'= 11°
33.3	silty clay (CH)	C'= 7.5 kPa φ'= 15°	------- -------
11.4	silty clay (CH)	C'= 35 kPa φ'= 18.5°	------- -------
32.4	silt with sand seams (ML)	C'= 7.5 kPa φ'= 35°	------- -------
Fraser Subdivision			
5.6	silty clay (CH)	C'= 50 kPa φ'= 16°	-------- --------
5.6	silty clay (CL)	C'= 65 kPa φ'= 17°	-------- --------
18.0	clayey silt (ML-CL)	C'= 10 kPa φ'= 34°	-------- --------

3.0 HISTORY OF SLOPE INSTABILITY

3.1 FACTORS AFFECTING SLOPE STABILITY

The CN North Division line passing through the Fraser Valley has many sites of continuing slope instability. The problems experienced at the Mile 30.8 site on the Tete Jaune subdivision are typical of many slide locations.

The original construction method employed during the 1914 construction of the line was a crude cut and fill technique utilizing the local lacustrine clay as embankment fill with little or no material preparation. The clay has a low remoulded strength and it is doubtful that any compaction techniques were employed in the original construction.

Consolidation settlements and the spreading action of fill embankment sections were offset by lifting and [redacted]allasting beneath the ties with crushed cinders. This surcharging of the crest of the railway embankments often aggravated the situation.

Construction and maintenance of drainage collection ditches upslope of the tracks increased stability through a general drawdown of the water table. Infrequent maintenance often reduced the effectiveness of this method and often resulted in accumulations of ponded water in blocked drainage ditches and high groundwater levels which further reduced stability.

The replacement of cinder ballast with crushed rock effectively increased the crest loading on embankments because of the higher specific gravity of the ballast. Higher crest loading also resulted from the switch in the mid-1950's from steam to diesel electric locomotives. Increased axle loading and increases in the number of cars per train has almost doubled the "traffic density", or annual accumulated gross weight of trains and goods travelling over the line since 1960.

The presence of alluvial fan and apron deposits interfingered with the overlying glaciolacustrine silts and clays often gives rise to artesian groundwater conditions. This mechanism, proposed by Thomson and Mekechuk (1982), can cause wide seasonal fluctuations in the stability of railway embankments. Figure 11 illustrates this mechanism schematically. Mekechuk (1981) also presents convincing evidence that the continuing long term trend of increasing annual precipitation in the central British Columbia region has been a significant factor in the reactivation of movements in quasi-stable slopes.

These factors have all contributed to the general decrease in embankment stability experienced over the North Division line throughout the Fraser Valley. Many sites with marginally-stable slopes have become areas of active settlement due to a combination of these factors.

3.2 ROTATIONAL SLIDE -- 1983

The small rotational slide which finally developed at the Mile 30.8 site in 1983 likely resulted from many of the long term influences combined with the unfavourable events of 1981 and 1982 which are discussed in the following paragraphs.

The long-term settlements and displacements resulting from increased embankment crest loading over a period of almost 70 years likely deformed the lacustrine clay beyond its peak strength. The post-peak strength reduction phenomenon reduced the stability of the embankment to the point where slow creeping of the soil mass occurred, resulting in continuous settlement of the tracks. This condition was aggravated during high rainfall seasons and periods of heavy rail traffic. Annual settlements in excess of 150 mm were reported by CN workers at this site, with a reported settlement of 300 mm in 1980 alone.

A small granular berm was constructed in 1981 near the crest of the slope, downslope of the rail line, in an ill-advised attempt to stabilize the slope. This resulted in further deterioration of rail trafficability and increased track settlements because the weight of the 1.5 metre thick fill effectively increased the driving forces at the crest of the slope.

Matters were made worse as 1982 recorded an unusually high rainfall as illustrated in Figures 12 and 13.

A geotechnical inspection in 1983 revealed the presence of a small rotational slide as illustrated in plan in Figure 2 and Plates 2 and 3.

The slide measured approximately 100 metres in width and 50 metres in length and was confined to a shallow depth. A visible headscarp was evident as well as several radial tension cracks and a well developed toe bulge. Small springs of water were seen ponding in the drainage ditch beside the toe bulge. A settlement of approximately 200 mm occurred in the track at the crest of the slope. Figure 14 illustrates the slide geometry in profile. A slight offset is visible in the line of telephone poles that traverses the site.

A site investigation and instrumentation program was conducted in the spring of 1984 to gather more information regarding the geometry of the sliding mass and the groundwater regime for use in design of remedial stabilizing works. A CN Rail geotechnical team conducted a site investigation consisting of 18 boreholes using CN Rail's track-mounted, hollow-stemmed auger drilling rig. A total of four slope indicators and 18 Casagrande-type standpipe piezometers were installed at the site as illustrated in Figure 15. The slope indicators were limited to the area of visible sliding instability to locate the seat of movement. Early results indicated the presence of a zone of concentrated downslope displacement approximately 7 metres below the ground surface. All four slope indicator

records locate this zone at approximately the same depth below the ground surface. Interestingly, this roughly corresponds to the inferred boundary between the brown and the grey silty clay sequences.

Many of the Casagrande-type standpipes remained virtually dry or registered water levels several metres below the ground surface. Although this seems to be in disagreement with the site observations indicating standing water in the drainage collector ditches, this phenomenon is explained in detail later in the report.

3.3 BACK-ANALYSIS OF 1983 SLIDE

The 1983 rotational slide at the Mile 30.8 site makes an excellent case history for back-analysis because the slide geometry and groundwater conditions are known in adequate detail.

The initial evaluation of shear strength properties operative at the time of sliding was conducted by the geotechnical staff at CN Rail's Edmonton office to provide input parameters to the design of remedial works. Subsequent computations were performed by the author to investigate the influence of varying the location of the groundwater table and the sliding surface as well as

inclusion of the crest loading contributed by the weight of passing freight trains.

The simplified slope geometry used in the initial analysis conducted by CN staff assumed a failure surface approximated by a series of circular arcs passing through the location of the sliding surface as inferred from the 1984 slope indicator profiles. The groundwater table is inferred from site observations of standing water pools at the upslope drainage collection ditch and the water present in the drainage ditch at the toe of the slide.

An effective stress analysis was used since the long-term stability of the slope was in question. An effective cohesion intercept of $C' = 0$ was chosen because the movements associated with the sliding mass had been occurring over a period in excess of 30 years, bringing the mobilized shear strength parameters to near-residual values. Limiting equilibrium slope stability analyses were conducted at the CN Rail Construction Office using a Simplified Bishop's Method computer program developed by CN staff. The results of this analysis yielded a factor of safety of unity for average mobilized shear strength parameters of $C' = 0$ and $\phi' = 13.25^{\circ}$. It should be noted that no allowance was made for the static or possible dynamic loading effects imposed by passing trains since these were judged to be of minor significance (the weight of the trains probably represents only 5% of the weight of the sliding mass).

These average effective strength parameters were then used in the design of the proposed remedial works.

Further limit equilibrium slope stability analyses were conducted by the author to investigate the influence of:

a) Varying the geometry of the sliding surface to an irregular shaped surface to approximate the observed field conditions more closely;

b) Varying the elevation of the groundwater surface to investigate the sensitivity of the analysis to the pore pressure distribution in the slope at the time of failure;

c) Inclusion of the static weight of the passing freight train on the crest of the slope.

The first two points are important because of the degree of uncertainty associated with the exact field conditions. The inclusion of the static weight of the passing trains in the analysis is thought prudent since settlements of the tracks during passage of trains has been observed at other sites of slope instability. Mekechuk (1981) reported observed settlements of 200 mm with the passage of a single freight train at a similar site. Since the total volume of material involved in the slide is relatively small, it is felt that the weight of the passing freight could play a small but significant role in assessing the shear strength parameters mobilized at the time of

sliding. Little is known about the dynamic loading effects imposed by the passing freight trains which may also have an influence on the slope stability of the site.

Figure 14 illustrates the simplified slope geometry complete with the phreatic surface located by site observations of standing water pools. The critical slip surface is developed from information gained from slope indicator profiles, observations of toe bulging, and the observed settlements of the railroad tracks.

Both the Morgenstern-Price and the Janbu Simplified methods of limiting equilibrium slope stability analysis were used to model the irregularly shaped failure surfaces. Long term stability analyses were performed using effective strength parameters . An effective cohesion intercept of $C' = 0$ was again employed due to the long history of movements documented at the site. The Morgenstern-Price computation yielded a factor of safety of unity for average mobilized shearing strength parameters of $C' = 0$ and $\phi' = 12^{o}$ with no train loading considered. The Janbu Simplified analysis indicated an average mobilized friction angle of $\phi' = 12.8^{o}$ for $F = 1.0$.

Parametric analyses were performed for varying phreatic surface conditions using the Morgenstern-Price analysis. Porewater pressure ratios, r_u varying from 0.0 (dry slope) to 0.5 (phreatic surface co-incident with ground surface) were input to produce the relationship between

factor of safety and water table elevation shown in Figure 16.

A probable range of r_u values, representative of groundwater conditions present at the time of the slide, of 0.2 to 0.4 produces a range of average mobilized friction angles between $\phi' = 10^o$ and 15^o at the time of sliding.

The final refinement to the analysis was the inclusion of a simulated dead load at the crest of the slope approximating the static load imposed by a passing freight train. The static load was evaluated based upon maximum axle loads of the locomotives passing over the section of track experiencing instability. The slope stability analyses indicated that inclusion of the static train load increased the required average mobilized shear strength by 7% and 11% for the Janbu Simplified and Morgenstern-Price analyses respectively to achieve a factor of safety of unity. Table 3 summarizes the results of the various analyses performed.

Consideration of the results of the various slope stability analyses performed indicates that the average effective shearing strength mobilized in the soil mass at the time of sliding likely lies between $\phi' = 11^o$ to 14^o. This compares favourably with the residual strength values obtained from direct shear testing of soil samples obtained during drilling at the Mile 30.8 site and also with CN Rail data obtained at other nearby locations. It is proposed that long term movements occurring within the slope over the

period of 70 years since original line construction have reduced the available soil strengths to near-residual values. This weakening of the soil combined with seasonally high groundwater levels is considered to be the cause of the failure of the slope.

It is interesting to note that the slope indicator records locate the seat of movement in close proximity to the inferred boundary between the upper brown silty clay and the lower grey silty clay. A slight stiffness contrast between these two units may have caused a preferential accumulation of deformation at this boundary as settlements and embankment creep took place. This mechanism may have contributed to the reduction in the shearing strength of the clays to near-residual values.

Also of interest is the fact that no artesian groundwater conditions were observed anywhere at the site. Artesian pore-pressure conditions are a common cause of slope instability along the Tete Jaune and Fraser Subdivision rail lines because of the widespread occurrence of submerged alluvial fan deposits. The cause of the Mile 30.8 slide can be explained satisfactorily without the presence of these conditions.

TABLE 3

SUMMARY OF SHEAR STRENGTH PROPERTIES BACK-CALCULATED FROM LIMITING EQUILIBRIUM ANALYSES FOR 1983 SLIDE

CASE	TYPE OF ANALYSIS USED	WITHOUT TRAIN LOAD	WITH TRAIN LOAD
1.	Bishop's Simplified	C'= 0 φ'= 13.25°	--------- ---------
2.	Morgenstern-Price	C'= 0 φ'= 12.0°	C'= 0 φ'= 13.3°
3.	Janbu Simplified	C'= 0 φ'= 12.8°	C'= 0 φ'= 13.7°

Note: The factor of safety at the time of sliding is assumed to be unity.

The elevation of the phreatic surface at the time of the 1983 slide was inferred from site observations of ponded water standing in the drainage collector ditches upslope of the rail line and the access road.

The effective cohesion intercept is assumed to be C'= 0 consistent with residual strength conditions.

4.0 REMEDIAL WORKS

4.1 DESIGN OF REMEDIAL WORKS

The design of remedial works for the Mile 30.8 site was conducted by CN Rail geotechnical staff in 1984 to correct the slope instability problem. Several alternative schemes were considered as corrective measures.

The possibility of relocating the tracks off the slide area was considered but rejected because it would introduce adverse curvature to the rail lines. The chance of triggering another slide in a new location also made this option unfavourable.

Drainage methods such as horizontal or vertical drains have been employed at several CN Rail sites in the Fraser Valley with varying degrees of success. It has often been found that areas underlain by alluvial fan deposits causing artesian porewater conditions can benefit from this type of drainage. This measure often proves to be temporary and requires periodic maintenance to prevent plugging of the drains. The Mile 30.8 site stratigraphy consists entirely of high plasticity silty clays which are particularly unsuited to this system of drainage.

Toe berms have proven to be effective long term, maintenance free methods of increasing slope stability at several CN Rail sites. The seat of movement at the Mile 30.8 site is relatively shallow and the failure surface daylights halfway down the slope. To build a stabilizing

berm midslope would likely create a second, potentially more dangerous slope failure triggered by the weight of the berm itself.

The volume of soil involved in the sliding mass is sufficiently large to make soil replacement, improvement, or reinforcement methods cost-prohibitive.

A satisfactory solution to the instability problem was developed as a combination of drainage methods incorporating counterfort drains and a stabilizing berm. The stabilizing berm was constructed midslope to arrest slope movements initiated at the crest. The counterfort drainage system was installed beneath the stabilizing berm. It consists of thirteen trenches running downslope. The trench drains effectively control the pore pressures in the soil beneath the stabilizing berm and prevent failure of the slope under the berm weight.

Figures 2, 3, and 17 illustrate the combination trench drainage/berm stabilization system in plan and profile views. The stabilizing berm was constructed by placing approximately 2.5 metres of free draining granular material on the access road, effectively lifting the roadway over the entire length of the site. The counterfort drains consist of a perforated drainage culvert installed in a 4 metre deep trench backfilled with coarse crushed stone and surrounded by a filter fabric.

The depth of the counterfort trenches was fixed by the maximum practical extension of a small backhoe and the

fact that increased depths would require substantial flattening of the sides of the excavation for safety reasons.

Since the trench drains could not be extended to a depth below the seat of sliding movement they contribute practically no buttressing effect to the sliding mass. Although this buttressing effect would be beneficial to the overall sliding stability of the soil mass it has little impact on the intended purpose of the drainage system, which was to control berm construction pore pressures and moderate seasonal groundwater fluctuations.

Some consideration was given to the possibility of placing drainage ditches parallel to the rail line, along the strike of the slope. This proposal was discarded because it could cause excessive settlements of the tracks by lowering the water table in the silty clay underlying the tracks. The length of such trenches would also increase the cost of the project considerably.

The design of the counterfort drains conforms closely to the practical approach suggested by Hutchinson (1977). A spacing of 20 metres centre to centre was adopted. The upslope ends of the drains are located approximately 25 metres downslope of the railroad tracks to prevent drawdown of the water table beneath the tracks and thus prevent further track settlement. The perforated culverts placed in the drainage trenches terminate at a collector pipe located approximately 80-100 metres downslope

of the rail line. The collector pipe channels the collected seepage to a stream at the toe of the slope. Generally, the quantity of seepage is small. Plates 5 through 6b document the trench drain and stabilizing berm construction.

Construction of the counterfort drainage system began in the spring of 1984 with the installation of the first six counterfort drains at the eastern end of the site.

An extensive instrumentation program was undertaken in the spring of 1985 prior to the completion of the counterfort drain installation and construction of the stabilizing berm in the summer of 1985.

4.2 1985 DRILLING AND INSTRUMENTATION PROGRAM

The aim of the drilling and instrumentation program conducted in the spring of 1985 was to install additional instrumentation to supplement the slope indicator and piezometric data being recovered from the site. The additional instrumentation consisted of a network of 55 pneumatic piezometers, 11 slope indicator casings, and 3 standpipes. The purpose of this additional equipment was to:

a) monitor pore pressure build-up during construction of the stabilizing berm for assessment of construction slope stability;

b) monitor the slope movements and identify the limits of the sliding mass;

c) study the influence of the counterfort drainage system on the pattern of groundwater flow;

d) assess the effectiveness of the corrective measures and attempt to optimize the design for future applications.

Figure 15 illustrates in plan the configuration of the completed instrumentation system.

The 1984 site investigation and instrumentation provided important information regarding the depth of the sliding surface and rough estimates of the phreatic surface location. It was generally discovered, however, that the majority of the Casagrande-type piezometers remained dry or indicated water levels well below that indicated by standing water ponded in surface drainage ditches. Further, these installations showed virtually no response to seasonal rainfall and runoff extremes indicating they may not be accurately reflecting groundwater conditions. For these reasons the decision was made to install pneumatic piezometers in the detailed 1985 instrumentation because of their rapid response times.

Piezometers were installed in a variety of different patterns to investigate different areas of interest. Many units were placed at the 7 metre depth to correspond to the location of the seat of sliding movement. Special patterns of instruments were installed between pairs of counterfort drains to detect the drawdown pattern induced by the drains. Some instruments were also placed outside the area of influence of the drains and outside of the area of mass movement to provide base values for comparison.

Instruments located beneath the proposed berm area gave important construction pore pressure information to guard against construction slope failure.

Nested piezometers and piezometers installed at different depths in closely spaced boreholes were situated to give some indication of porewater pressures at different depths. This information was correlated to determine the presence of flow gradients or conversely hydrostatic phreatic conditions. Silt lenses and other interesting features were often selected as piezometer installation locations.

Boreholes were advanced using an Acker hollow-stem auger drilling rig mounted on a self-propelled tractor unit. Soil samples were recovered using a 75 mm (3 inch) diameter split spoon sampler and extruded on site for visual classification, photographs, and classification testing. Samples were periodically obtained using 75 mm (3 inch) diameter Shelby tubes for subsequent laboratory tests.

Since the 1984 drilling program had included detailed sampling to identify the stratigraphy at the site, the 1985 program concentrated on rapid installation of instrumentation in features of interest. Instruments were installed over a large area on a roughly rectangular grid to provide a detailed look at the groundwater flow patterns over the entire site of instability.

Slope indicators were generally installed in boreholes augered to a depth of 12-13 metres (40 feet) below ground level and backfilled with a uniform coarse-grained sand to minimize casing deflections due to consolidation of the backfill around the casing.

A typical pneumatic piezometer installation is illustrated in Figure 18. The general installation procedure involved augering to the depth of interest, followed by sampling through the hollow stem augers with either the 75 mm diameter split spoon sampler or Shelby tubes. The pneumatic piezometer tip was then lowered into the hole left by the sampler and backfilled with coarse sand to the top of the 75 mm diameter hole. Approximately 0.6 metres of bentonite pellets were then added, followed by a bucket of water to start the sealing process. The remainder of the borehole was then backfilled by hand with natural clay cuttings from the borehole. A plastic riser pipe was placed at the top of each borehole to raise the piezometer lines approximately 1.5 metres above ground level to protect the cables against damage from animals. These pipes were

subsequently raised or extended in areas where fill placement raised the ground level.

Most instruments were tested for pore pressure development within one day of installation. The majority were found to have equalized to steady state conditions by this time.

4.3 MONITORING OF INSTRUMENTATION

A regular schedule of site visits was conducted to gather periodic instrument readings and to record site developments that effected instrument behavior. Site visits were conducted every two weeks during high rainfall and runoff periods and less frequently during the winter months.

Site observations were recorded including such information as stages of berm and counterfort drain construction, quantity of flow in counterfort drainage culverts and drainage ditches, and appearance of ponded water on or near the site.

The monitoring process involved the reading of the pneumatic piezometers using a portable pressure gauge. The pneumatic piezometers were Petur Model P-100 instruments with single intake and bypass tubes. Casagrande-type standpipe piezometers were read using an electronic dip meter on a less frequent basis because the water levels in

these instruments showed little fluctuation with time. Slope indicator profiles were prepared periodically to determine the presence and magnitude of any slope displacements.

Occasionally site visits involved the repair of damaged instruments. Some slope indicator casings were damaged when fill was compacted around them during berm construction. Several pneumatic piezometer cables were chewed through by small animals during the winter months. Snow banks formed around the conductor pipes, allowing the animals access to the exposed cabling.

The pneumatic piezometer installations proved to be extremely reliable over the two year study period. Only two of the 55 instruments installed were confirmed to be defective during this period. All other instruments appeared to be operating properly during the study period.

4.4 IN SITU FALLING-HEAD PERMEABILITY TESTING

Falling-head permeability tests were performed in eight of the Casagrande-type standpipe piezometers in the summer of 1986.

The tests were conducted approximately two years after the installation of the piezometers to ensure that steady state conditions prevailed .

When the Casagrande-type piezometers were installed the porous tips were placed in sand-filled pockets in the bottom of 200 mm (8 inch) diameter dry-augered boreholes. The boreholes were sealed with a bentonite plug and backfilled with natural clay. The tips ranged in depth from 3.7 metres to 20 metres below the ground surface.

The purpose of the in situ permeability testing was:

a) to give approximate estimates of the range of in situ permeability at various locations and depths at the site;

b) to yield information that would assist in the evaluation of the principal permeability contrast (the ratio of K_1/K_2)

c) to yield information that would assist in the evaluation of the continuity of jointing, fissuring,

and silt lensing that may contribute a favourable drainage mechanism or secondary permeability to the soil.

The in situ permeability tests were conducted by filling a portion of the riser pipe attached to the piezometer tip with water and measuring the rate at which the water level dropped with an electronic dipmeter.

It is interesting to note that the water levels in the standpipes returned to their previous levels at the completion of the test. This provides confirmation that the standpipe piezometers are in fact indicating the true phreatic conditions.

The reduction of the field data and interpretation of in situ permeability from the falling-head tests was performed following a variety of guidelines including those proposed by Hvorslev (1951), Gibson (1963), Al-Dhahir and Morgenstern (1969), and Premchitt and Brand (1981).

Hvorslev (1951) formulated the theoretical time-lag equalization process that allows the interpretation of in situ permeability from falling-head tests conducted in standpipes. The original theory applied only to instruments installed in incompressible soils. Premchitt and Brand (1981) concluded that Hvorslev's theory will also give a good approximation of soil permeability in compressible soils if the system flexibility is small. Low values of system flexibility result from conditions where the

piezometer system has a low stiffness and the soil is relatively incompressible. System flexibility, denoted by the dimensionless parameter μ, is interpreted from the values of t50 and t90, representing the values of the time required for 50% and 90% pore pressure equalization respectively. Figure 19 (Premchitt and Brand, 1981) illustrates the relationship between the ratio of t90/t50 and the system stiffness parameter .

The values of μ interpreted from the falling-head test data range from 0.0 to 3.0 . This means that the Hvorslev theory will yield reasonable approximations of the coefficient of in situ permeability.

Table 4 presents the results of the falling-head tests.

It should be noted that the interpretation of in situ permeability is affected by the ratio of the principal permeabilities. K_1/K_2. The influence is relatively small compared to other uncertainties inherent in the test, it is important to note that the actual groundwater flow distribution is more a function of the ratio of the principal permeabilities rather than their absolute values.

The spread in the permeability values predicted by the tests likely results from a combination of the following factors:

a) If it is assumed that the distribution of jointing, fissuring and silt lensing is uniform over the site and

with depth, the range of permeability values could reflect the fact that some instruments connect to these features while others do not.

b) The range of permeability values may accurately reflect the true in situ conditions of the lacustrine deposits. The deposits may have no features which would impart a secondary permeability, implying that the range in permeability results from differences in material composition across the site.

c) Unknown factors such as possible smearing of the borehole walls, faulty bentonite seals, siltation of the porous tip or leakage from the riser pipe could contribute to the variability of the results.

TABLE 4

INSITU FALLING HEAD TEST RESULTS

INSTRUMENT #	DEPTH (m)	INTAKE FACTOR	T_{50} (s x 10^4)	T_{90}	T_{90}/T_{50}	μ	K_h (m/s)
84-01	18.1	2.01	110	240	2.2	0	$3x10^{-10}$
84-03	19.5	2.64	150	290	1.9	0	$2x10^{-10}$
84-05	17.9	3.06	68	145	2.1	0	$4x10^{-10}$
84-07	19.5	2.64	2	36	16.0	3	$5x10^{-9}$
84-14A	7.3	2.43	78	200	2.6	0	$1x10^{-10}$
84-14B	4.0	2.64	2	8	4.7	0	$3x10^{-9}$
84-17A	7.9	2.64	0.2	0.9	5.0	0.1	$3x10^{-8}$
84-17B	4.0	2.64	20	115	5.8	0.2	$3x10^{-10}$

4.5 SLOPE INDICATOR RESULTS

A network of slope indicator casings was installed at the Mile 30.8 site to monitor the extent and magnitude of slope movements. Figure 15 illustrates the layout of the slope indicators including four casings installed to approximately 20 metre depths in 1984 and eleven casings installed to 13 metre depths in 1985.

The casings installed in 1984 were concentrated in the area where the small rotational/translational slide was discovered in 1983. The slope indicator profiles obtained between March 1984 and April 1985 indicated the presence of a zone of concentrated displacement located approximately 6.7 to 7.6 metres below the ground surface. These profiles also indicated that the sliding movements were continuing until the spring of 1985 when berm construction began. Figure 20 illustrates a typical profile from instrument SI84-4.

Eleven additional slope indicator casings were installed over the extent of the site in the spring of 1985 to identify the limits of the slope movements. Installation took place in a three week period between May 15 and June 7, 1985. Berm construction followed between July 8 and August 17, 1985. The majority of movement indicated on these slope indicator profiles is attributable to settlement of the granular backfill in the boreholes surrounding the casings.

The most recent profiling, conducted on May 7, 1986, indicates the cessation of slope movements. A zone of concentrated displacement which may be associated with the 1983 slide was found in one of the 1985 slope indicator casings located at the northwestern corner of the site at a depth of approximately 6 metres below the ground surface. This zone of movement does not appear on the other instruments installed in the area and may represent a limited area of movement.

Track settlement over the 250 metre length of the site may not stem from displacement along a single well-defined slip surface as is the case in the area of the small rotational slide. The fact that the stabilizing berm was completed within two months of the installation of the 1985 slope indicator casings may indicate that slope movements were curtailed by the berm construction and were never detected. The settlement of the tracks could have resulted from the movement of a number of smaller sliding masses. In any case, track settlements appear to have been successfully halted by the placement of the restraining berm.

5.0 ANALYSIS

5.1 METHODOLOGY

The extensive instrumentation installed at the Mile 30.8 site was originally designed to monitor the performance of the counterfort drainage system and assess the stability of the modified slope. It also represented a unique opportunity to study the influence of berm loading, drainage measures, and seasonal rainfall and runoff variations on the groundwater flow regime.

In order to accomplish all of the above-mentioned tasks it was necessary to analyze the groundwater data using a number of different approaches. The first and perhaps most obvious approach was to plot the pore pressure data coming from the field on slope cross-sections taken parallel and perpendicular to the dip of the slope. Pore pressure data obtained before and after the drain installation provided a good method of assessing the effectiveness of the drains in regulating the elevation of the phreatic surface. This method also allows the easy determination of drawdown profiles between counterfort drains and allows quick evaluation of slope stability for different phreatic conditions.

A second useful method of analysis was the plotting of instrument phreatic levels against time. Many factors that may influence the phreatic conditions at each instrument can be superimposed on the plot of phreatic

elevation versus time to see if any direct influence can be inferred from the pattern of fluctuations. Some factors superimposed were the schedule of berm construction, total precipitation including rain and snowfall and mean daily temperatures occurring at the site.

The final method of analysis used was a comparison of the pore pressure distributions determined from the piezometers and standpipes to those predicted in numerical analyses and recorded case histories in the literature.

The insight gained at each level of analysis helped to produce a clear, integrated understanding of the response of the groundwater regime to varying environmental and loading conditions. This is a necessary step in understanding the contribution of varying groundwater conditions to slope instability.

5.2 ANALYSIS OF CROSS-SECTIONS

Several cross-sections were taken through the slope both parallel and perpendicular to the dip. The locations of the sections was determined to provide a maximum representation of piezometric data and are illustrated in Figure 21.

Figure 22 illustrates the sections taken parallel to the dip of the slope. These show the position of the

track and siding centerlines, the original slope geometry and the final geometry after berm placement, instrumentation location and location of the counterfort drain inverts. Also plotted are the maximum annual phreatic levels for 1985 and 1986. Both maxima occurred in the spring season. The phreatic levels were determined by converting pore pressures measured in the pneumatic piezometers to an equivalent water column.

Figure 23 illustrates cross-sections taken along the strike of the slope. These sections show the location of the original and final ground surfaces and the position of the perforated culvert at the bottom of the counterfort drains. Annual maximum phreatic levels are also plotted for 1985 and 1986.

In both types of cross-sections it is important to recognize the boundary between areas where the counterfort drains were installed at different times. Drains #1 through #6 were placed in the summer of 1984, and drains #7 through #14 were installed one year later, in the summer of 1985.

OBSERVATIONS

Some general observations can be made after studying the groundwater data plotted on the cross-sections.

1) The elevation of the phreatic surface is generally higher for the spring of 1986 than the spring of 1985 (Figure 22). This could be caused by a number of factors including increased precipitation and runoff, excess pore-pressures left over from the berm-building construction, or a disruption of free drainage caused by trench excavation. Most likely the rise in the phreatic surface results from a combination of these effects.

2) The influence of the counterfort drains in lowering the phreatic surface begins approximately 20 to 25 metres downslope from the beginning of the drains (Figure 22).

3) The phreatic surface appears to be successfully lowered near or below the level of the drainage culvert inverts in the sections taken through the area of 1984 drain installations (Section D - Figure 23). In the area where counterfort drains were installed in 1985 the drainage does not seem to have much effect (Section E - Figure 23). There may be a time lag between counterfort drain installation and lowering of the phreatic level as new drainage paths are

established. This could explain the apparent effectiveness of the 1984 drains as opposed to the 1985 drains.

4) The phreatic profile is generally parallel to the ground surface except where influenced by the counterfort drains (Figure 22).

5) Phreatic levels indicate the presence of standing water in the drainage ditches on the upslope shoulder of the access roadway, at some locations (Figure 22). This is consistent with site observations.

6) The standpipes installed in sand-filled boreholes on the upslope shoulder of the track ballast (units SP85-56, SP85-57, and SP85-58 in Figure 15) generally indicate water levels consistent with observed levels in the drainage ditch. The Casagrande-type piezometers, however, indicate water levels considerably below those indicated by surrounding pneumatic piezometers. These sealed Casagrande-type tips are generally installed at greater depth than the pneumatic piezometer tips. The discrepancy in phreatic levels could result from the presence of a downward flow gradient, perched water table conditions, or from some irregularity in the groundwater conditions caused by unusual stratigraphic features.

A downward flow gradient could result if the lacustrine deposits were underlain by a high permeability

aquifer which acted as a drain to the less pervious overlying deposits. There is some evidence to suggest the presence of a submerged alluvial fan deposit underlying the site but this was never intersected in the boreholes. The presence of some wet swampy areas upslope and downslope of the site suggests that these areas are not being drained by a high permeability underlayer. It is also well documented at other CN Rail sites in the area that artesian groundwater conditions often result when underlying alluvial fan deposits are present. No artesian pressures have been observed at the Mile 30.8 site. Thus the available evidence would not support the draining underlayer theory.

There is some evidence that supports the perched water table theory as a possible explanation of the discrepancy in phreatic levels predicted by shallow vs. deep installations. The existence of the failure surface at approximately the interface level between the two separate clay units identified in the site stratigraphy suggests a possible contrast in structure between these units. This contrast in material structure could result in two separate water tables in the clay units. It is possible that groundwater flow entering the overlying unit through infiltration may flow downslope approximately parallel to the ground surface and not impact significantly upon the deeper water table.

Since no concrete evidence is available to resolve the question, both mechanisms are presented as possible

explanations for the phenomenon. It is clear, however, that porewater pressures at depth in the slope are less than hydrostatic (Figure 25c,d). This has positive implications in terms of increased slope stability.

7) The drawdown profile between successive drains is relatively flat, as illustrated in Figure 23. The phreatic level is generally closer to the level of the drain inverts in the section stabilized by 1984 drains than in the section stabilized by 1985 drains. This again suggests that there is a time lag between drain installation and full effectiveness. The flat drawdown profile between drains suggests a relatively high horizontal to vertical permeability ratio.

8) The rise in the level of the phreatic surface due to berm construction is substantially greater in the area stabilized by 1985 drains compared to the area of 1984 drains (Figure 23). An average of 2 metres of fill was placed on the roadway to act as a stabilizing toe berm for the upper slope. This resulted in an average increase in the phreatic level of 0.6 metres and 1.7 metres for the 1984 drain area and the 1985 drain area respectively (or $\overline{B}$ values of approximately 0.16 and 0.39 respectively). This again indicated that the 1984 drains were functioning more efficiently.

9) Some pneumatic piezometers located at depth indicate phreatic levels lower than those installed to shallow depths (Figure 25c). This again may indicate either downward flow gradients or perched water table conditions.

10) A re-analysis of the sliding stability of the slope using the Morgenstern-Price limiting equilibrium technique yields a factor of safety of F = 1.45 for the new slope geometry. The residual strength parameters of $\phi' = 12^{o}$ and C' = 0 developed from back-analysis of the 1983 sliding movements and the maximum water table elevation determined during the 1985 season were used in the analysis.

5.3 TIME RESPONSE OF GROUNDWATER CONDITIONS

Piezometric data obtained from pneumatic piezometers and Casagrande-type piezometers are plotted against their period of record to facilitate comparison of piezometric level fluctuations to environmental influences. The superposition of rainfall and temperature data on the same time scale reveals interesting correlations of groundwater response to seasonal rainfall and runoff patterns.

The Mile 30.8 Tete Jaune Subdivision site is located approximately equidistant from the McBride North and Dunster meteorological recording stations. Both stations

record similar rainfall and temperature trends. The data recorded at the McBride North station are used in this study because a greater period of record is available.

Figures 12 and 13 illustrate the precipitation history at the McBride site in the form of total annual precipitation (including snowfall) from 1923 to 1986, and average monthly precipitation from 1923 to 1986 respectively. Figure 24 gives a more detailed record of monthly precipitation from May 1985 to August 1986 (roughly the period of piezometric observations).

Figure 24 illustrates a plot of the average monthly temperature over the period of piezometric record (May 1985 to August 1986).

Piezometric data is plotted against time for the period between May 1985 and August 1986. Instruments are plotted in groups depending upon their locations on the site and the type of installation. These groupings include the following:

a) instruments located at the crest of the slope outside of the area of berm construction and counterfort drainage;

b) instruments located midslope in the area stabilized by drains installed in 1984;

c) instruments located midslope in the area stabilized by drains installed in 1985;

d) instruments located upslope of the rail lines (approximately 20 metres);

e) Casagrande-type standpipe piezometers and standpipes installed at various locations on the site;

f) multiple-tip pneumatic piezometer installations.

Figures 25a through 25d illustrate the piezometric data obtained for each of the above-mentioned groups. The elevation of the phreatic surface is inferred by converting pore-pressure to equivalent height of water column.

Also plotted on these Figures are the dates corresponding to the beginning and completion of berm construction. All Figures, including those illustrating the average monthly rainfall and temperature, are plotted on the same time scale for easy comparison.

OBSERVATIONS

The following general observations are offered based upon the analysis of the time-dependent groundwater fluctuations.

1). A study of the total precipitation records illustrated in Figure 24 indicates peak precipitation periods in September-October, March, and June-July. Temperature data indicates that average daily temperatures fell below 0^{o} C between November 4, 1985 and February 23, 1986. Therefore, groundwater level increases in September/October, March and June/July would be anticipated due to rainfall. It would also be expected that the period from November to March would be a period of static or declining phreatic levels, with a resurgence in March due to spring thaw. Therefore, the following cyclic groundwater level fluctuations may be predicted:

June - July, 1985	Increasing
August - September	Static or decreasing
September - November	Increasing
November 1985 - March, 1986	Static or decreasing
March - July	Increasing
July - September	Static or decreasing

2) The construction of the stabilizing berm took place between July 8 and August 17, 1985. Pore pressure increases were expected in instruments located in the area covered by fill. Counterfort drain trenches were excavated for drains #7 to #14 between June 11 and July 6, 1985. A decrease in

pore pressure could be expected in some piezometers located in this area.

Items #1 and #2 above outline the anticipated groundwater response to environmental and construction influences. The following observations are made on actual instrument response.

3) Pneumatic piezometers installed upslope of the rail lines, outside of the zone of sliding movements, did not respond sharply to rainfall or runoff influences (Figure 25b, bottom). Instruments registered a gradual lowering of the phreatic level during the dry summer months, showed no response to September/October rainfalls, and remained virtually static during the winter period. The only significant rise in phreatic levels occurred during high spring rainfall in June/July.

The relative insensitivity of these instruments to environmental effects may be partially explained by the fact that these instruments are located approximately 30 metres back from the crest of the slope where groundwater levels are less subject to fluctuation

4) Instruments located adjacent to the rail line at the crest of the slope showed good response to rainfall fluctuations (Figure 25b, top). A clear trend of lowering of the phreatic surface is evident during the dry summer

months and over the winter period. Sharp increases in the phreatic level mark the high rainfall periods in September/October and June/July. Some instruments also responded to the March rainfall and snow runoff . It can be expected that groundwater levels at the very crest of the slope are the most variable, as is illustrated by the piezometric records.

5) The pneumatic piezometers installed in the area stabilized by 1984 counterfort drains show excellent response to berm construction during the period from July to mid-August (Figure 25a). There is virtually no response to high rainfall occurring in the June/July, March, and September/October periods. A gradual decline in pore pressures during the winter and the dry summer months is also evident.

The absence of rainfall response illustrates the effectiveness of the counterfort drains in damping groundwater response to seasonal rainfall and runoff fluctuations. The elimination of these seasonal peaks is very important for slope stability reasons because high groundwater levels reduce the effective sliding resistance of the soil mass.

6) The pneumatic piezometers located in the area stabilized by 1985 counterfort drains also show excellent response to the 1985 berm construction (Figure 25a). The

magnitude of the registered pore pressure increases is substantially greater than that registered in the area of 1984 drains. There is no response to high rainfall in September/October and mixed response to the March and June events. Some instruments showed significant pore pressure decreases during the winter months and the dry summer months.

The response of the piezometers in the area stabilized by 1985 counterfort drains indicates many of the same benefits observed in the area of 1984 drains. The greater pore pressure response to berm building and the mixed response during the March and June rainfall maxima indicates that the drainage measures may take some time to become effective. The success of the 1984 drainage trenches is encouraging and hopefully this same level of efficiency will be achieved over the site.

7) The Casagrande-type standpipe piezometers show very little response to seasonal rainfall and runoff fluctuations (Figure 25d). Many of these instruments are located at greater depths than the pneumatic piezometer tips. Pore pressure fluctuations at depth are smaller because of the increased distance to the infiltration boundary at the ground surface. Many Casagrande-type tips also have a 25 mm diameter riser pipe leading to much increased response lag-time.

The standpipes installed in sand boreholes adjacent to the railroad tracks showed some response to spring rainfalls but this was likely due to their close proximity to the drainage ditch which often fills with water.

5.4 LITERATURE REVIEW AND CASE HISTORIES

A review was conducted of the available literature documenting groundwater flow in hillsides and the influence of counterfort or trench drains on groundwater pore pressures. Various researchers have published both numerical groundwater flow models and illustrative case histories pertaining to the transient flow conditions present in hillsides with and without artificial drainage features.

The review is divided into two main sections: the determination of the natural groundwater conditions, developed for varying stratigraphies and permeability conditions, in response to seasonal rainfall and runoff fluctuations; and investigation of the influence of counterfort and trench drainage on groundwater pressure distributions. The information gathered from the literature was compared to the groundwater observations at the Mile 30.8 site to attempt to explain observed patterns.

A comparison was also made between the observed groundwater pressure patterns and the patterns predicted by finite element method and closed form analytical solutions presented in the literature.

GROUNDWATER FLOW IN SLOPES

The following generalized groundwater flow trends were gathered from the available literature:

Thomson and Mekechuk (1982) proposed a mechanism for the infiltration of precipitation and spring runoff into the sloping lacustrine sediments. It was proposed that the alluvial fans and aprons prevalent along the sides of the river valley were submerged by the later lacustrine deposits. The interfingering of discontinuous sand and silt laminations throughout the clay mass provided a direct connection to these large granular aquifers and thus a good conduit for passage of groundwater. It is also likely that these granular aquifers underly the lacustrine deposits at depth. At the Mile 30.8 site the appearance of an alluvial fan upslope from the rail lines suggests this possibility.

Freeze and Witherspoon (1967) report that the presence of an underlying high conductivity zone has the effect of making the entire slope a recharge area. This in turn results in downward flow gradients in the slope.

Instrumentation installed at the Mile 30.8 site has indicated the possible presence of these downward gradients at different locations on the site. The findings of Lafleur and Marcotte (1984) support this hypothesis and also suggest the presence of upward gradients at the toe of the slope as this area constitutes a discharge zone. These upward gradients can have serious implications with regard to the stability of the slope as this reduces effective stresses in the resisting mass at the toe, and thus the available shearing resistance (Hodge and Freeze, 1977 ; Lafleur and Marcotte, 1984). No upward gradients were observed at the toe of the slope at the Mile 30.8 site, but they may exist nevertheless .

The seepage that was observed at the drainage ditch upslope of the access roadway indicates that this minor depression is a small discharge zone. This is consistent with the findings of Freeze and Witherspoon (1967) and Selim (1975). These researchers indicate that hummocky terrain often results in a complex groundwater flow pattern based upon the superposition of a number of local sub-basins upon the major groundwater flow basin. Minor topographic highs and lows become areas of recharge and discharge respectively. This would explain the fact that discharge was observed at this trench in the middle of the slope while the majority of the slope illustrates a general

downward flow gradient in response to the major groundwater basin.

Two distinct groundwater flow regimes have been identified in a number of investigations in clay slopes. These systems are characterized by an upper zone that responds dramatically to seasonal infiltration due to rainfall and runoff; and a lower zone that is uninfluenced by seasonal precipitation fluctuations (Lafleur and Marcotte, 1984 ; Lau and Kenney, 1984). This lower unit begins at approximately 7 metres below the ground surface. It generally exhibits steady groundwater pressures and its response is governed by the proximity of high conductivity aquifers, permeability anisotropy, the overall geometry of the drainage basin, and the boundary conditions present at the margins of the basin.

Weeks (1969) provided evidence supporting this theory in a slope which contained solifluction lobes. These lobes essentially represent masses of soil which have failed due to high groundwater pressures following deglaciation. Weeks discovered that piezometers installed above the plane of movement responded dramatically to rainfall fluctuations, while instruments installed at a greater depth responded slowly to inflow from proximal aquifers. The influence of the shear surface developed during mass movement is largely unknown but is thought to act as a partial barrier to groundwater flow.

The Mile 30.8 site exhibits this two-level phenomenon to some degree. Both pneumatic piezometers and Casagrande-type standpipes installed at depth illustrate a subdued response to seasonal precipitation fluctuations, while the response of shallow instruments correlates strongly with precipitation records. Interestingly, the seat of the 1983 slope movement lies approximately 7 metres below the ground surface. Groundwater information was not available in sufficient detail to determine the presence of any boundary between high and low response zones.

Permeability anisotropy is considered to be the general rule in stratified deposits and is discussed in some detail by Maasland (1957). A detailed investigation into the ratio of maximum permeability (generally parallel to stratification) and minimum permeability (usually perpendicular to stratification) for varved deposits is documented by Kenney and Chan (1973). This study indicates a relatively low permeability ratio of less than 5. The Mile 30.8 stratigraphy consists of thick sequences of mottled clay with discontinuous silt laminations. This likely results in some permeability anisotropy because the silt layers are more conductive than the general clay matrix.

Morgenstern and Sangrey (1978) report that even minor permeability ratios can exert a significant influence on pore pressure conditions. Hodge and Freeze (1977) found

that the orientation of the principal permeabilities also plays an important role in determining pore pressure distributions. It was found that the highest hydraulic pressure heads, and consequently the worst case pore pressures from a slope stability viewpoint, developed when the maximum conductivity was oriented parallel to the sloping ground surface. Borehole data obtained at the Mile 30.8 site indicates that the orientation of the silt lenses and the boundary between the two clay layers lie parallel to the sloping ground surface (Figure 8). This has also been observed at other CN Rail sites in the Fraser Valley (Mekechuk, 1981).

Changes in the groundwater pressure regime are produced by changes in the boundary conditions of a drainage basin. These can take the form of changes in applied stresses or changes in hydraulic conditions at the ground surface (Kenney and Lau, 1984). Stress changes can result from placement or removal of soil from the ground surface, for example, in the form of stabilizing berms or excavations. Changes in rates of precipitation and infiltration, or changes in the size or geometry of the recharge and discharge areas are possible hydraulic changes the system can undergo.

Decreases in piezometric levels during the winter months results from decreased infiltration due to freezing of the ground surface. This change in hydraulic conditions

at the ground surface results in consolidation of the soil during the winter months. Resumption of infiltration in the spring results in increasing groundwater pressures and hence swelling of the soil. Kenney and Lau (1984) also reported that the consolidation process occurring during the winter months took place at a much reduced rate compared to spring increases in pore pressure.

These effects are well pronounced at the Mile 30.8 site. In addition, changes in stress boundary conditions were met with predictable changes in groundwater pressures during trench excavation and berm construction.

Hydrostatic groundwater conditions assumed in many limiting equilibrium slope stability analyses are often unconservative. The presence of upward gradients at the toe of slopes is more detrimental than the static groundwater conditions generally assumed (Lafleur and Lefebvre, 1980). This condition does not appear to be present at the Mile 30.8 site. The downward gradients that have been observed in some areas in the slope will actually increase slope stability over assumed hydrostatic conditions.

COUNTERFORT DRAINS

Counterfort drainage techniques have been successfully employed in remedial slope stabilization projects for over 100 years. Early designs required the placement of the granular-filled drainage trenches to a depth below the inferred level of the slip surface. An increase in slope stability was thus achieved through a lowering of the pore pressures on the failure surface and an increase in shear strength due to the buttressing effect of the strong granular material (Henkel, 1957 ; Hutchinson, 1977).

It is now generally realized that trench drains which do not intersect the seat of slope movement still provide a beneficial lowering of the pore pressures on the failure surface, and thus increase slope stability. Stanic (1984) suggests that the redirection of flow away from the dominant downslope movement can also be helpful to slope stability. The change in the hydraulic boundary conditions causes a flow of water towards the drains, moving approximately perpendicular to the dip of the slope. This redirection of flow, if it becomes established, can be beneficial in terms of slope stability even if no permanent lowering of the overall groundwater level occurs. In practice this effect is very difficult to quantify.

Trench or counterfort drains often exhibit a distinct moderating influence on groundwater level fluctuations. Seasonal rainfall and infiltration fluctuations generally produce large variations in groundwater levels and are therefore hazardous from a slope stability viewpoint. Trench drainage systems have a damping influence on the magnitude of groundwater level peaks resulting from seasonal rainfall extremes (Lafleur and Marcotte, 1984 ; Weeks, 1969 ; Hutchinson, 1977). This effect was particularly pronounced at the Mile 30.8 site in the area stabilized by drains in 1984.

The contrast between the effectiveness of the drains placed in 1984 and the relatively minor influence of the drains placed in 1985 suggests a time lag between drain installation and full effectiveness. Henkel (1957) suggests that some time lag is involved as the clay between the drains consolidates under the influence of the new vertical 'zero head' boundary conditions introduced by the drains. They also report that actual time to equilibration to steady state seepage conditions is much shorter than that predicted by traditional oedometer test results. This is likely due to the presence of structural features in the clay which give it a secondary permeability and the three-dimensional aspects of drainage.

Hutchinson (1977) describes the changes in the groundwater pressure regime as a rapid pressure drop due to

stress reduction during excavation, and subsequent swelling of the clay between and around the drains. This is then followed by some consolidation of the clay as steady state seepage conditions are re-established with time. Stanic (1984) suggests that this process can take place in less than 6 months, but this is largely dependent upon the type of soil present and the presence of secondary drainage features such as fissures or lenses.

The efficiency of the counterfort drainage trenches at the Mile 30.8 site appears to have improved one year after initial installation and may be expected to continue to improve with time as new drainage paths are established.

Observations by Weeks (1969) and Symons and Booth (1971) on groundwater response to a counterfort drainage system installed in a stiff fissured clay deposit indicate that the influence of the drains is not fully developed until some distance (3.5 times the drain spacing in the reported case) downslope from the drain heads. They also reported that the drain's influence on groundwater conditions quickly reduced to zero at a short distance (0.3 times drain spacing) downslope of the drain ends. At the Mile 30.8 site full drawdown occurs at approximately 25 metres (1.25 times drain spacing) downslope of the drain heads (Figure 22). No instrumentation was installed downslope of the the drain ends.

Drain influence will vary considerably over the length of the installation in response to non-homogeneity of the soil conditions (Hutchinson, 1977).

Weeks (1969) also reports that piezometers installed below a remnant slip surface registered no average decline in groundwater pressures after installation of the counterfort drains. These instruments did, however, show a stabilization of groundwater levels. This situation indicates the presence of a perched water table, consistent with the two level theory presented by Lafleur and Marcotte (1984). Instruments located in the lower unit will not respond dramatically to fluctuations in water levels in the upper unit.

Deep Casagrande-type standpipe piezometers installed at the Mile 30.8 site showed little response to counterfort drain installation (Figure 25d). These instruments are located well below the failure plane associated with the 1983 slide and are founded within the lower clay unit. This unit corresponds to the zone showing little response to infiltration and it is reasonable that this unit would also show little response to drain installation as it behaves as though it were separated from the overlying material

5.5 GROUNDWATER FLOW MODEL CONFIGURATION

A more accurate description of the actual groundwater conditions present at the Mile 30.8 site could be obtained through the use of a finite-element or finite-difference flow simulation model. An iterative approach involving the comparison of numerical modelling results with observed groundwater patterns could be used to further explore the relationship between natural and construction influences on the groundwater conditions. The following section contains an outline of the features required in the numerical model and the boundary conditions necessary for flow simulations at the site.

The complexity of the Mile 30.8 site would make the use of a three-dimensional (3-D) model advantageous because of the variation in site conditions in plan and profile. Although the site stratigraphy is not overly complex, the presence of the counterfort drainage trenches results in the need to model groundwater flow variance in plan as well as profile. A 3-D finite-difference groundwater flow model such as that developed by McDonald and Harbaugh (1985) would be a good choice.

A useful starting point, prior to attempting the 3D modelling, would be to begin with a coarse-meshed, two-dimensional (2-D) model of the Fraser River valley as illustrated in Figure 26. This 2-D model could be used to explore the influence of varying boundary conditions and

simplifying assumptions on the regional flow basin. The more detailed 3-D model grid would represent only a small part of the larger slope profile used in the 2-D studies. The boundary conditions used in the 3-D model could then be developed from element or nodal information from the 2-D model simulations.

Both the two and three dimensional models selected for use in the study should be transient-flow models because the site groundwater levels appear to be sensitive to seasonal precipitation fluctuations. The precipitation hydrograph, modified for periods of sub-zero degree celsius temperatures and spring runoff, should be input as infiltration at the ground surface.

The groundwater flow models should be capable of modelling confined and unconfined flow to enable the simulation of perched water table and free surface conditions. The ability to simulate flow between a surficial, unsaturated soil zone and an underlying saturated soil zone is also desirable as this will more accurately represent the actual moisture transfer process occurring in the soil profile.

The slope stratigraphy and the soil hydraulic properties are known to a limited extent. Borehole data indicate the presence of silt lenses, within the clay matrix, which dip towards the center of the valley. No continuity of lensing was found during the drilling investigations, but the relatively flat drawdown profiles

between the counterfort drains indicates that the lenses may contribute to a horizontal permeability that is higher than the vertical permeability. The ratio of principal permeabilities is unknown but the absolute values of permeability, as determined by field falling-head permeability tests, are contained in Table 4.

The presence of a high-permeability granular layer underlying the site at depth was never confirmed in the borehole investigations. The potential influence of this and other uncertain features could be investigated in some detail with a parametric study of soil hydraulic properties and stratigraphic features.

Comparison of modelling results and the reported field observations will lead to an improved understanding of the groundwater flow and pore-pressure distribution patterns. It may also prove to be a useful way of optimizing the configuration of future slope stabilization works.

6.0 CONCLUSIONS

1) Slope movements and the resultant track settlements were documented at the Mile 30.8 site for over thirty years. These movements were induced by increasing weight of railway ballasts, increasing rail traffic density, and periodic high groundwater levels. This movement, combined with the consolidation settlements occurring after rail line construction combined to create a weakened zone in the lacustrine clay. The operative shear strength in this weakened zone was reduced to a near-residual value.

2) No evidence was found to indicate the presence of a remnant slide at the site.

3) High rainfall in 1982 and an ill-advised berm placement in 1981 led to high pore-pressures and eventual failure of the slope. Failure took place in the form of a small rotational/translational slide with a small headscarp, toe bulge, and radial tension cracks.

4) Back-analysis of the rotational slide failure using limiting equilibrium techniques yielded effective mobilized shearing strength parameters consistent with those developed in residual strength direct shear tests conducted on samples obtained from the site. In this analysis the static loading at the slope crest contributed by the passing freight trains

was considered to represent a small but significant influence and was included in the computation. The dynamic loading effect is difficult to define and was not included in this analysis.

5) Remedial stabilization works consisted of building a toe reinforcement berm and flattening of the steeper portions of the slope. An extensive counterfort drainage system was installed in the slope beneath the stabilizing berm to control the construction pore pressures and lower the groundwater levels in the slope. An extensive network of slope indicators and pneumatic piezometers was installed to monitor the slope performance and verify the design assumptions.

The remedial works have been successful in eliminating slope movements and track settlements.

6) The installation of the counterfort drainage system has eliminated groundwater peaks caused by seasonal fluctuations in rainfall and runoff. The drains were also effective in reducing the construction pore pressures caused by berm placement and in accelerating the consolidation process following construction.

7) The influence of the counterfort drains appears to increase with time after installation as the drainage paths become re-established after construction. The drains became

effective in drawing down the water table approximately 25 metres downslope from their upslope heads.

8) A very flat drawdown profile was established between drains, indicating a high ratio of horizontal to vertical permeability. This likely results from the secondary permeability contributed by the discontinuous silt and sand lenses, and possible fissuring.

9) The nested pneumatic piezometers and the Casagrande-type standpipe piezometers indicate that groundwater conditions at the site are not hydrostatic. Deeper instruments locate the phreatic surface at greater depth than shallow instruments. This could be caused by either downward flow gradients or by perched water table conditions. No conclusive evidence is available to confirm either hypothesis, but superficial evidence supports the perched water table explanation.

10) Although artesian groundwater conditions are often associated with buried fan deposits in the Fraser Valley, no artesian conditions were found at this site. There is evidence to suggest the presence of a buried fan deposit at the site which, if present, likely acts as a partial drain for the overlying lacustrine sediments, rather than causing artesian pore pressures.

11) There appears to be an upper clay layer that exhibits dramatic groundwater response to rainfall and infiltration fluctuations. The soil beneath this layer shows less response to the rainfall fluctuations and exhibits more uniform groundwater pressures. The upper unit is likely limited to approximately 5-10 metres in thickness.

7.0 RECOMMENDATIONS

Since the construction of the remedial works at the Mile 30.8 site there have been no reports of rail traffic disruption or slope movements. Based upon field performance and instrumentation data the combined counterfort drainage and stabilizing berm appear to have halted the slope movements and reduced the magnitude of seasonal ground water level fluctuations.

Discussions with CN Rail engineering personnel indicate that the combined counterfort drain/stabilizing berm system costs only slightly more than a conventional berm retaining structure while offering significant benefits. These include control of construction pore pressure development and reduction in seasonal ground water level fluctuations. The combined drainage/berm system appears to offer a reliable long-term, maintenance free solution to slope stability problems at the Mile 30.8 site and has potential applications at other sites of slope instability. This system could be particularly effective in areas where slope movements are initiated by ground water level fluctuations caused by seasonal rainfall and runoff extremes.

There exists a need for continued monitoring of the slope indicators and piezometers at the site to ensure the effectiveness of the remedial works in stabilizing the slope. Continued monitoring of the ground water conditions would also be useful in determining whether the beneficial influence of the counterfort drains continues to increase with time.

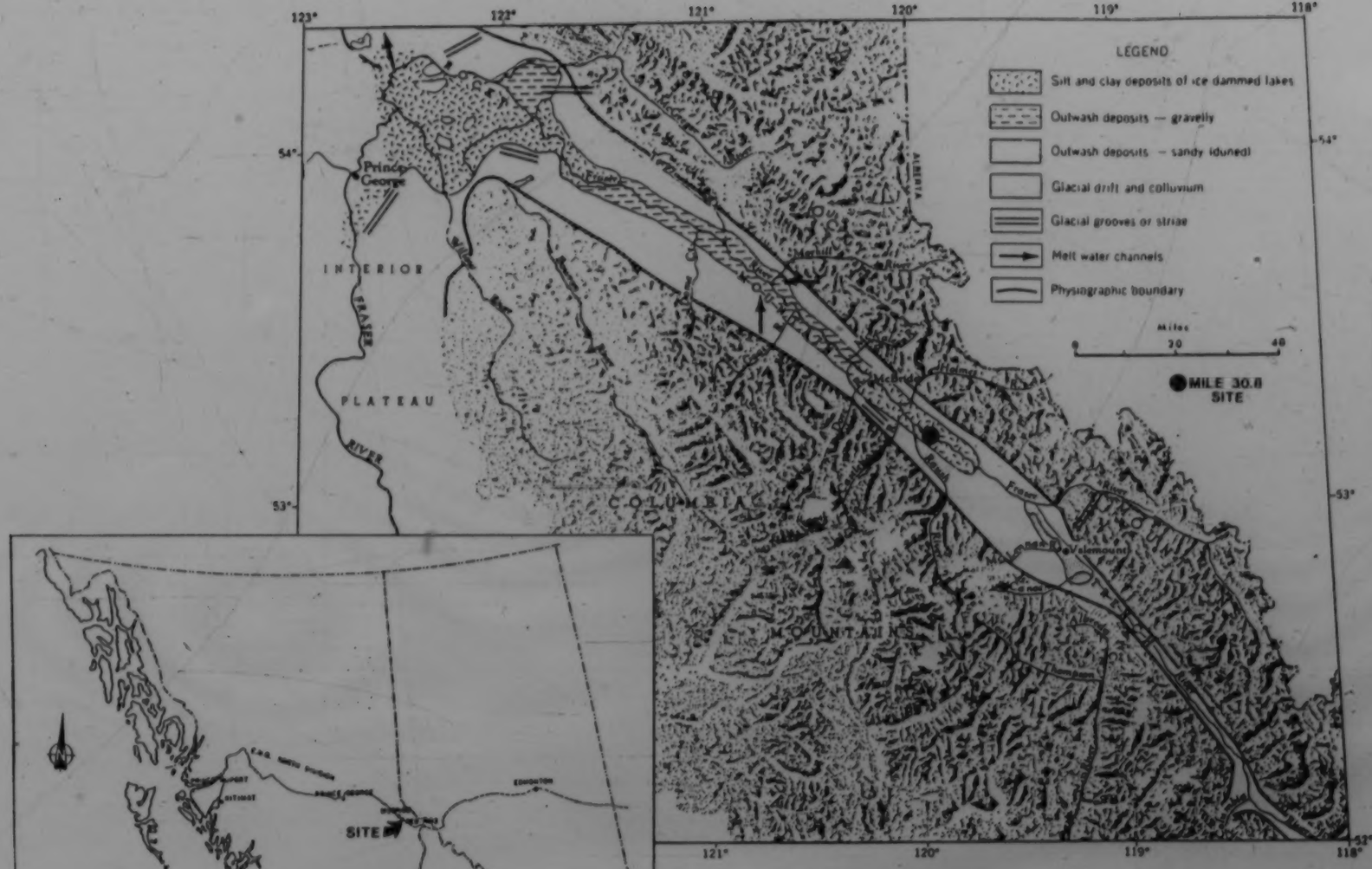

Figure 1. Glacial geology and physiographic divisions in the upper part of the Fraser Valley (modified from Hortie et al, 1970)

Inset: CN Rail North Division in central British Columbia (modified from Mekechuk, 1981)

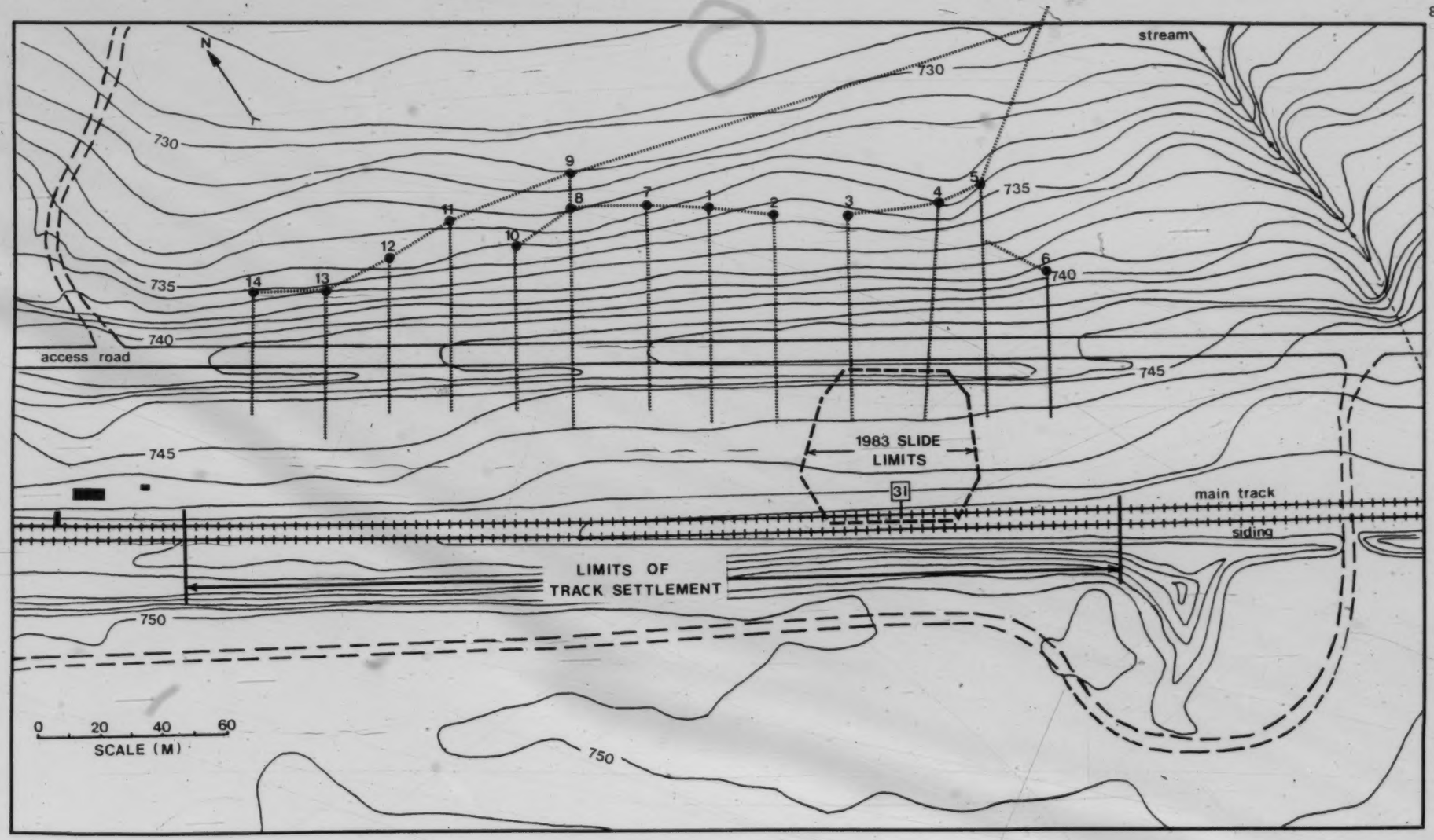

Figure 2. Site plan view illustrating the limits of the slope movements and the location of the counterfort drains.

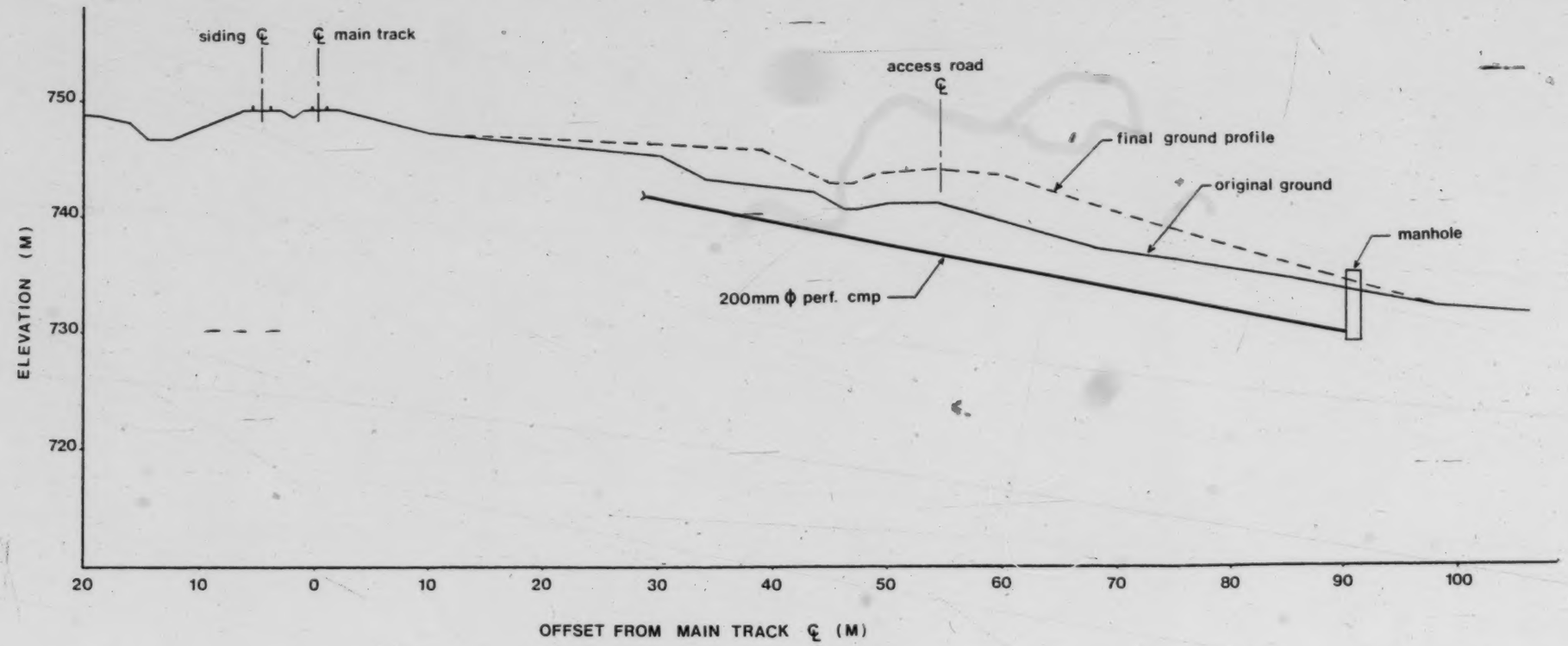

Figure 3. Typical slope cross-section at Mile 30.8 site.

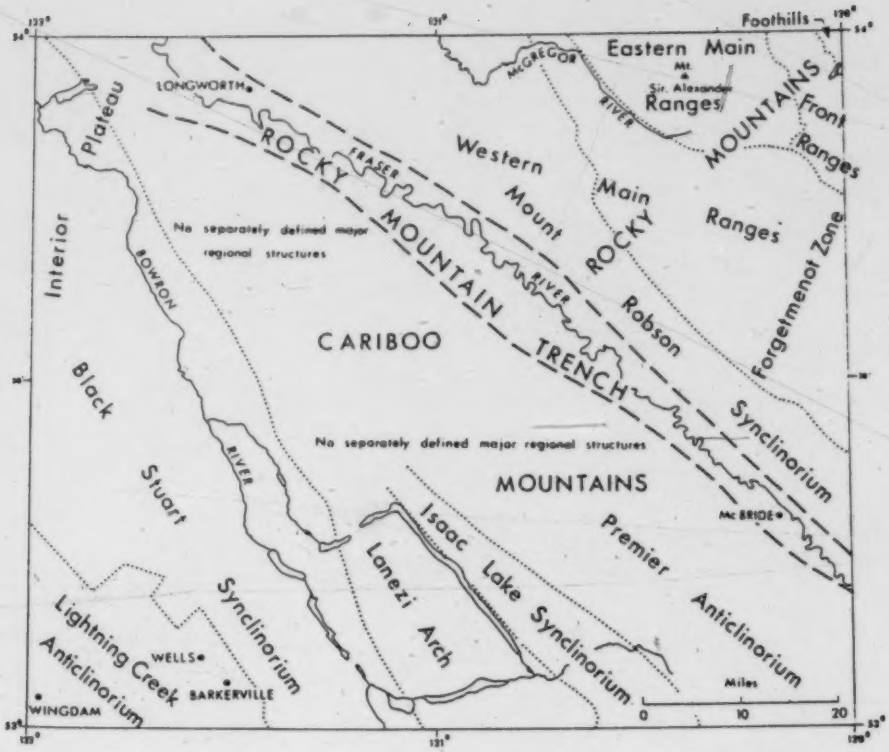

Figure 4. Structural elements and physiographic divisions in McBride map-area (from Campbell et al, 1973)

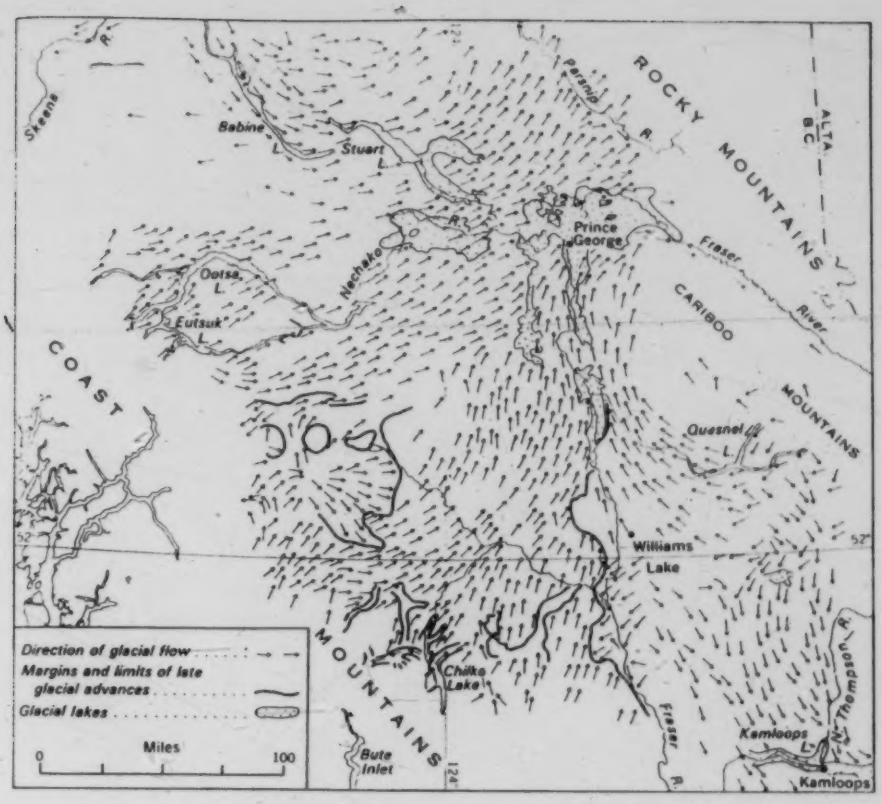

Figure 5. Direction of ice movements in central British Columbia of Fraser ice sheet and late ice readvances (from Tipper, 1971).

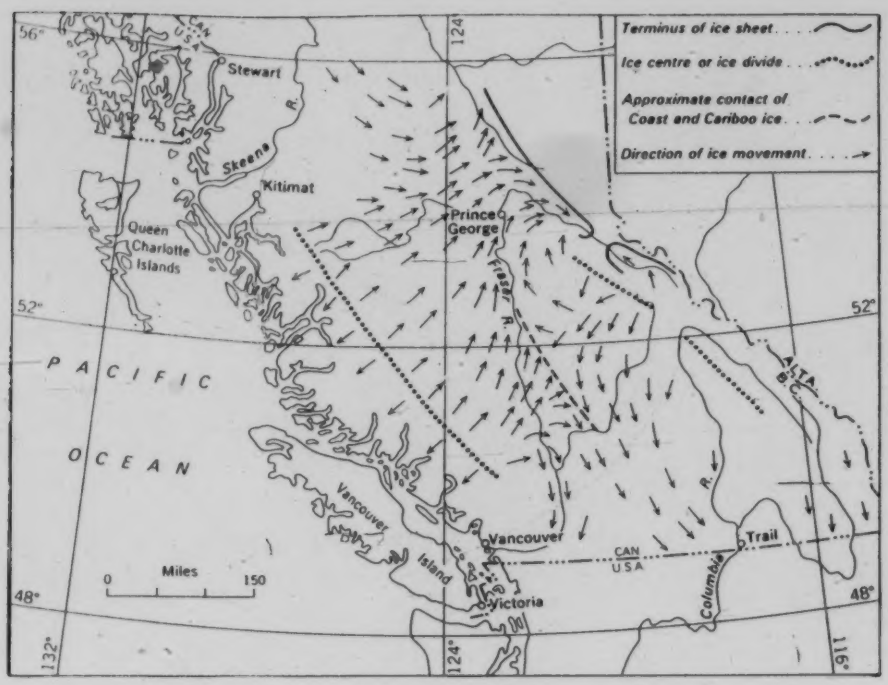

Figure 6. Maximum extent of Fraser ice sheet in central British Columbia (from Tipper, 1971).

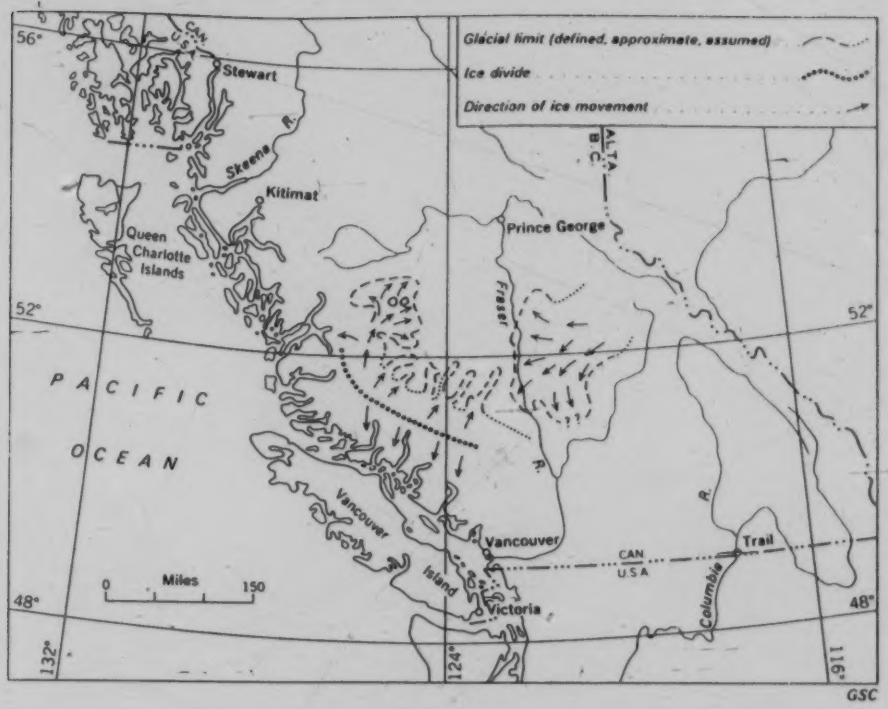

Figure 7. Maximum extent of late ice readvance in central British Columbia (from Tipper, 1971).

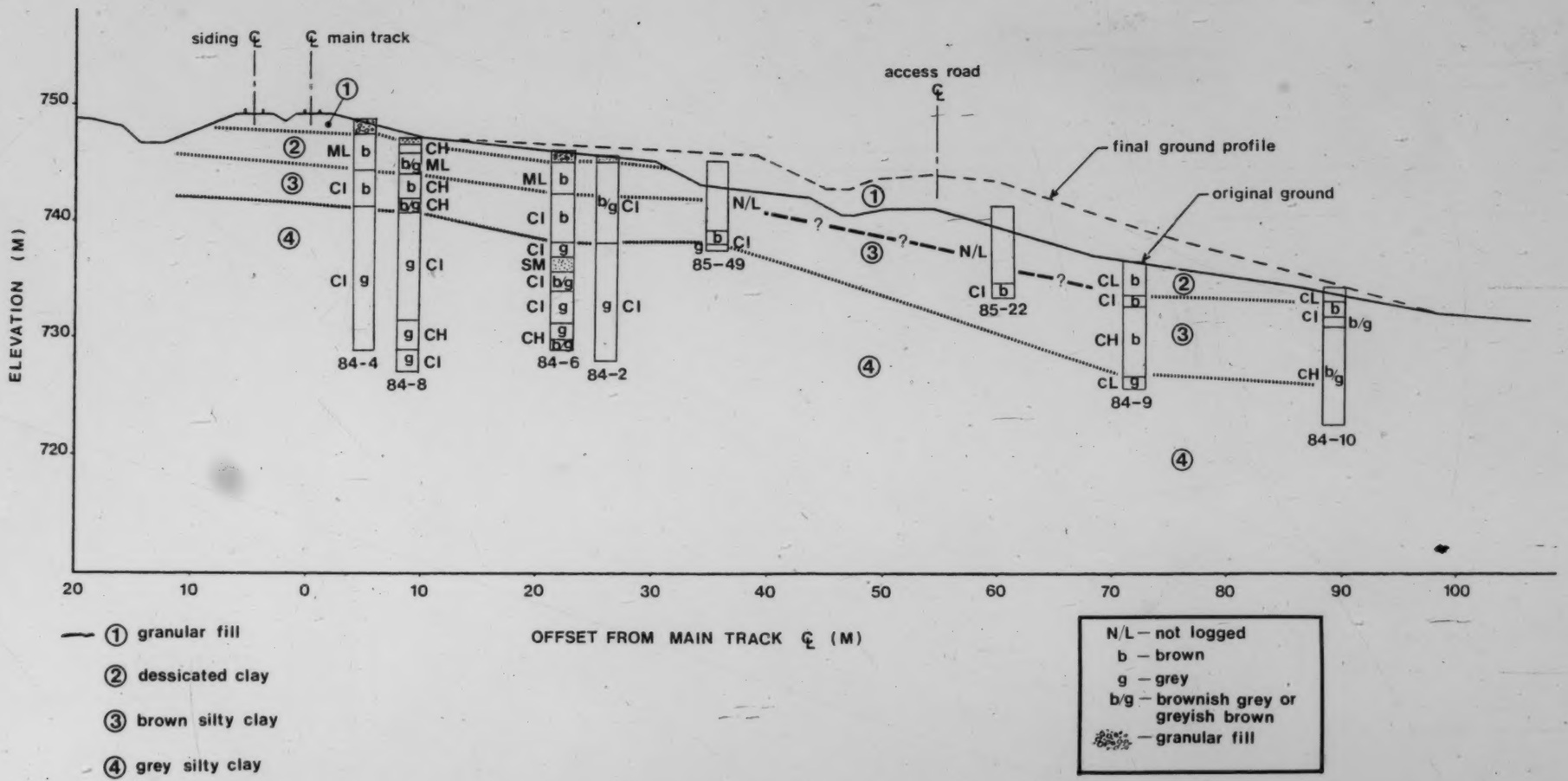

Figure 8. Geologic cross-section developed from borehole data.

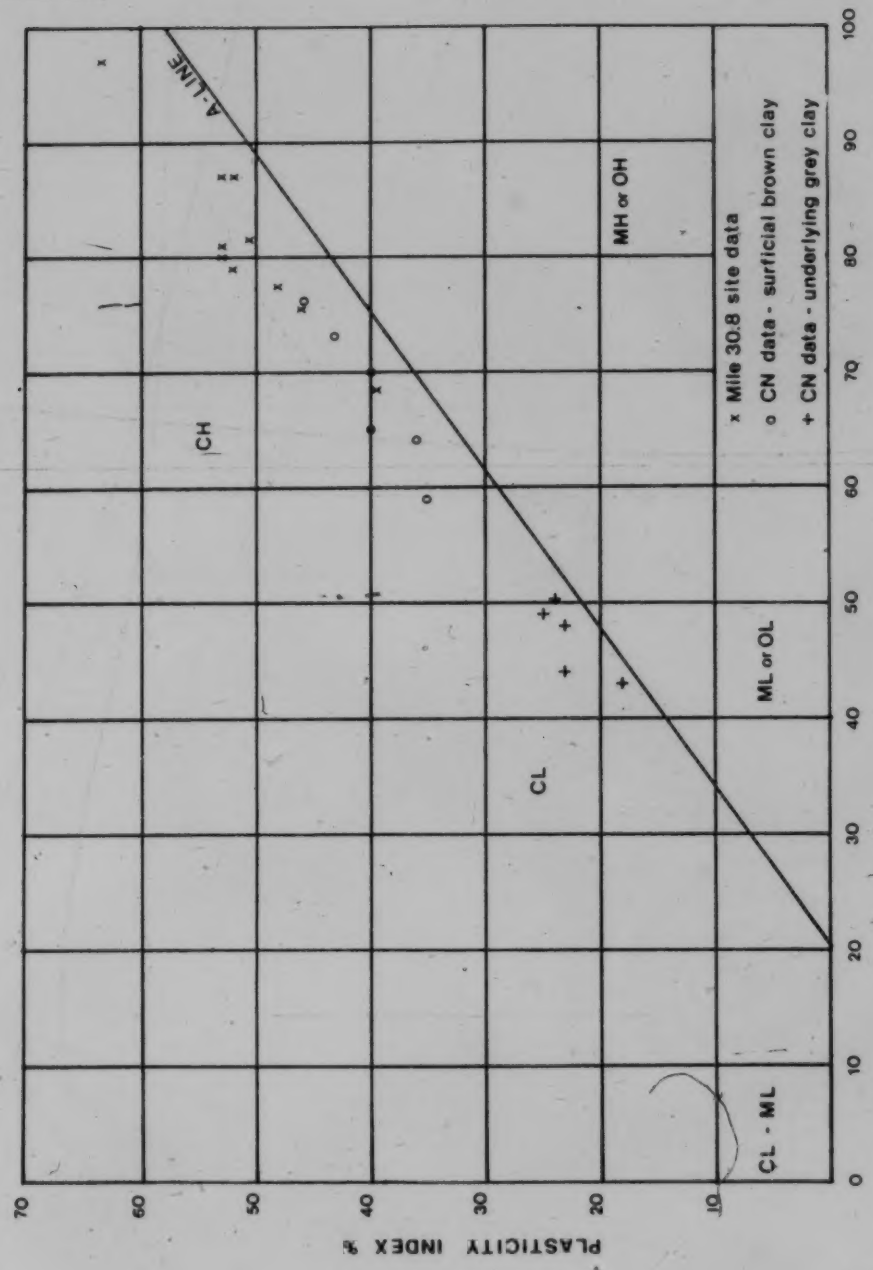

Figure 9. Plasticity chart for lacustrine deposits at the CN Rail Mile 30.8 site and the surrounding area.

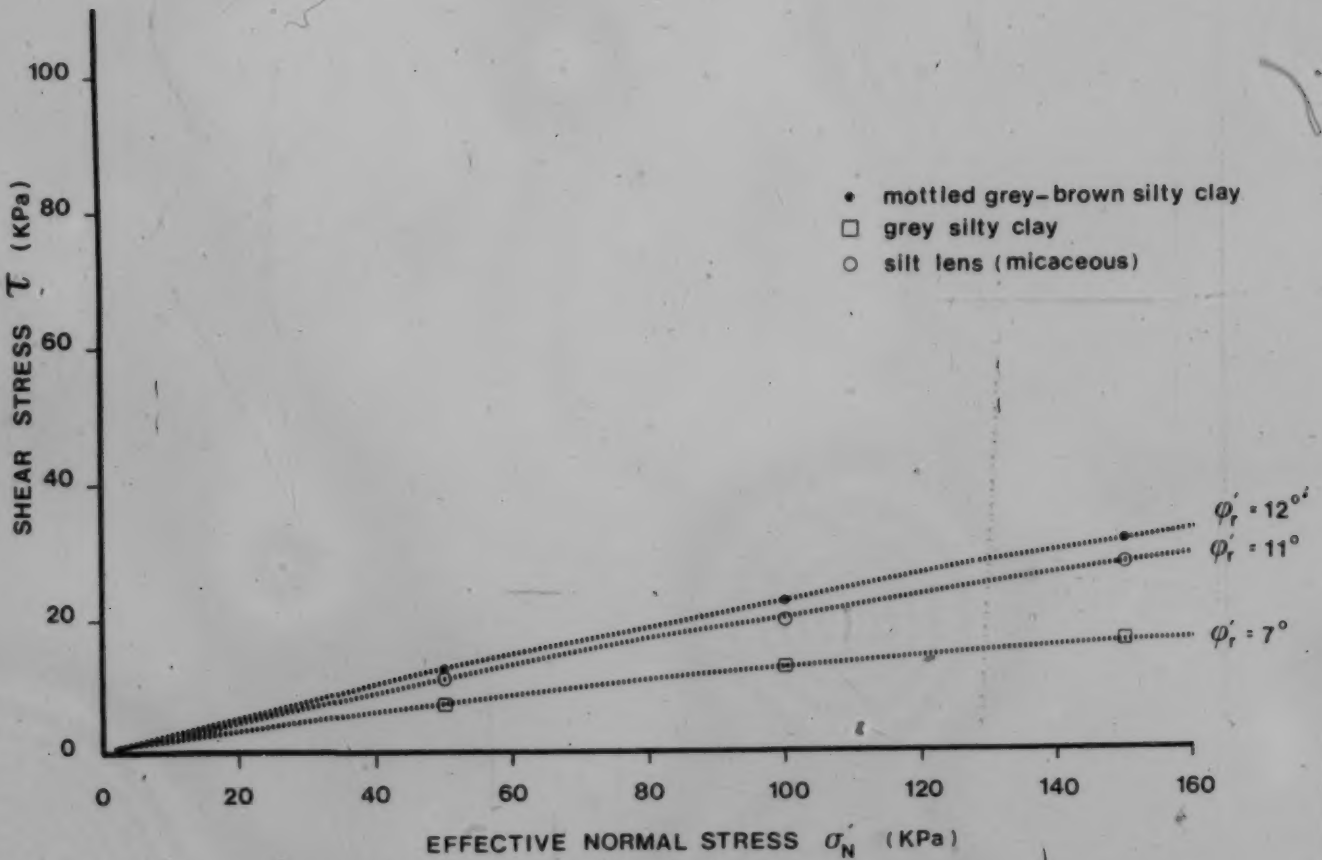

Figure 10. Residual shearing strength envelopes developed from multi-stage direct shear testing of lacustrine clay samples obtained from the Mile 30.8 site.

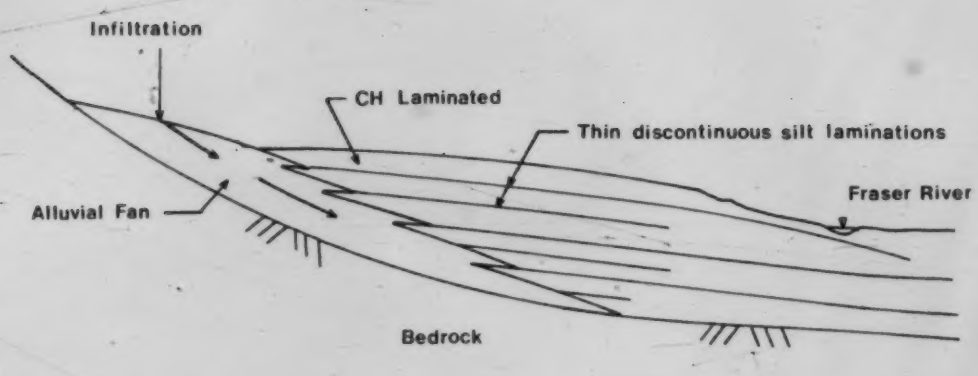

Figure 11. Schematic cross-section illustrating water infiltration mechanism (modified from Thomson and Mekechuk, 1982).

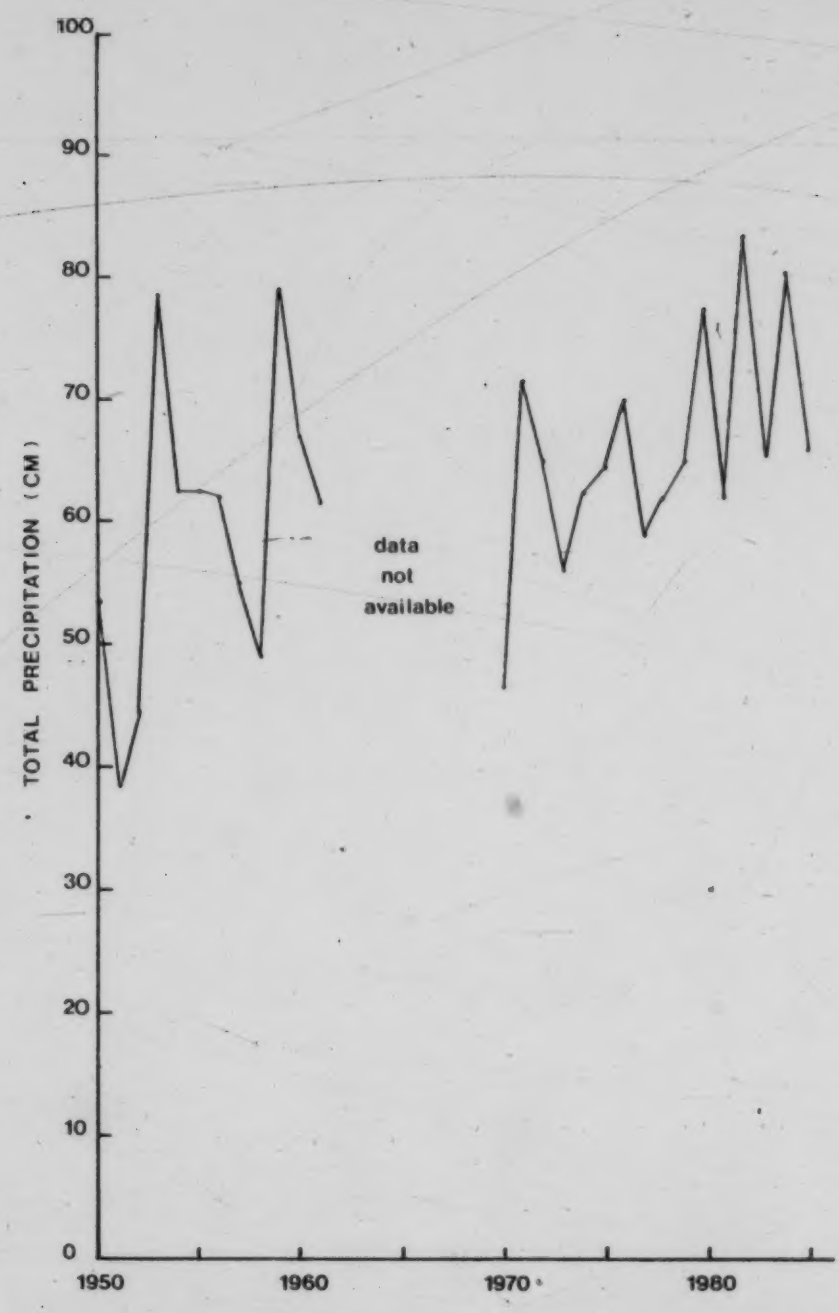

Figure 12. Total annual precipitation since 1950 measured at the McBride North meteorological station.

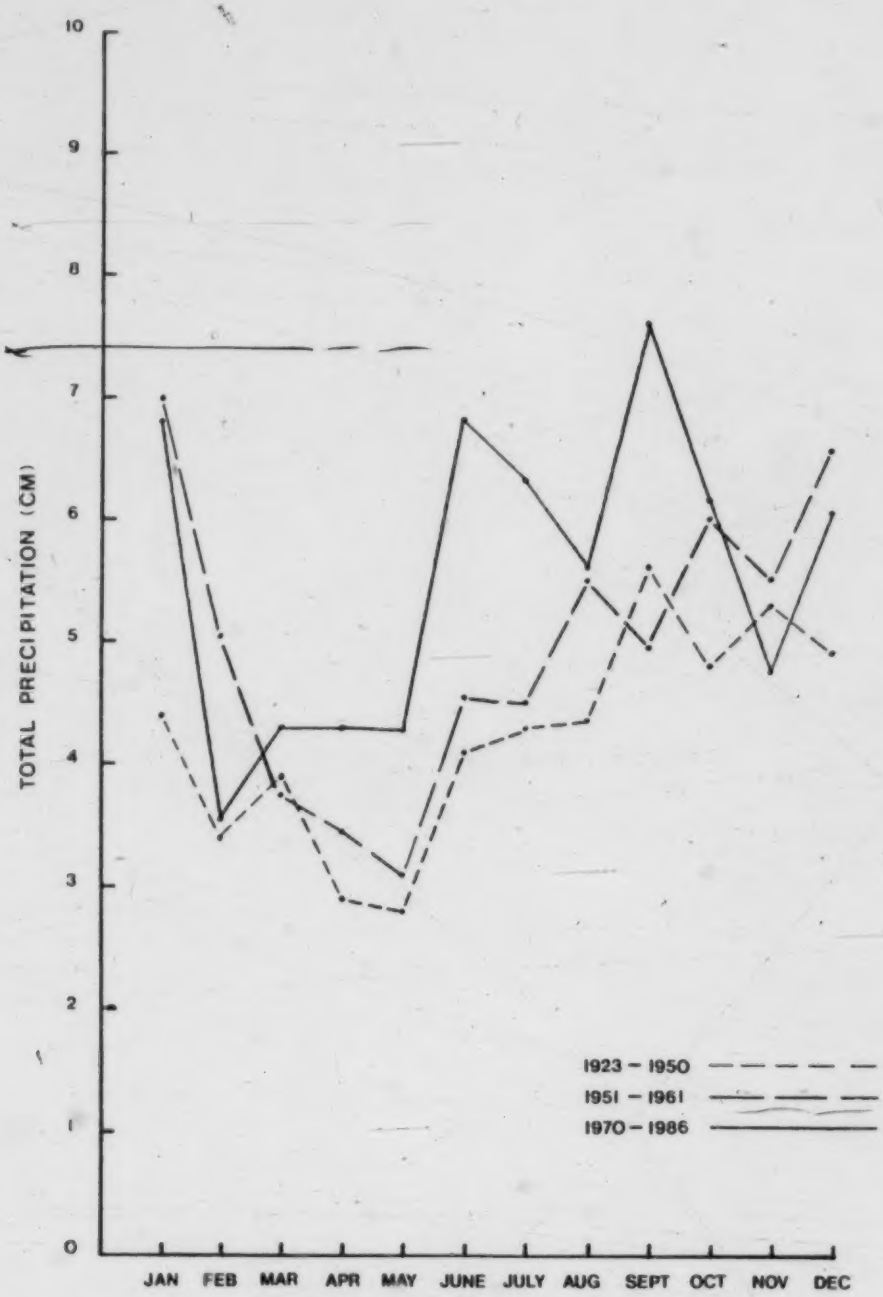

Figure 13. Average monthly precipitation for various periods between 1923-1986 for McBride, B.C. (modified from Mekechuk, 1981).

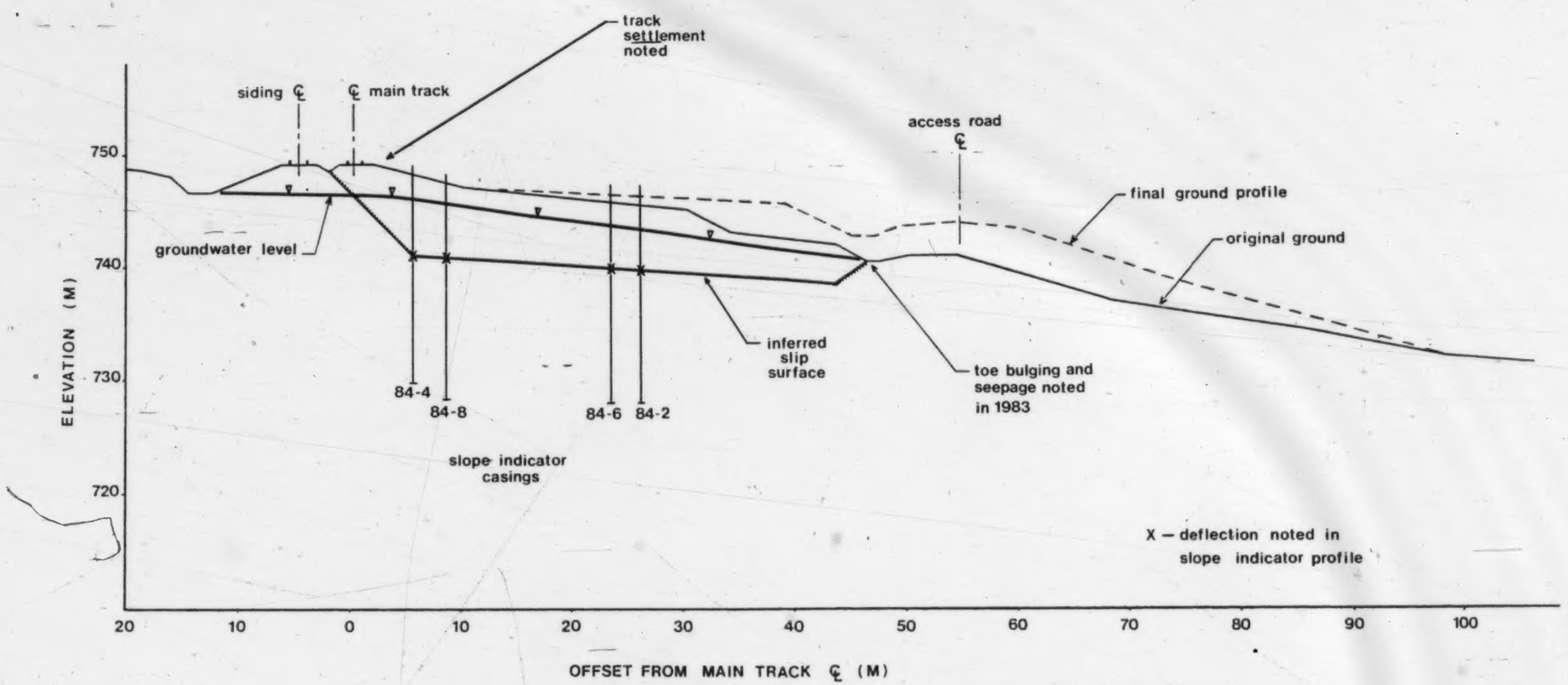

Figure 14. Slope profile illustrating the 1983 slide geometry as inferred from site observations and slope indicator profile records.

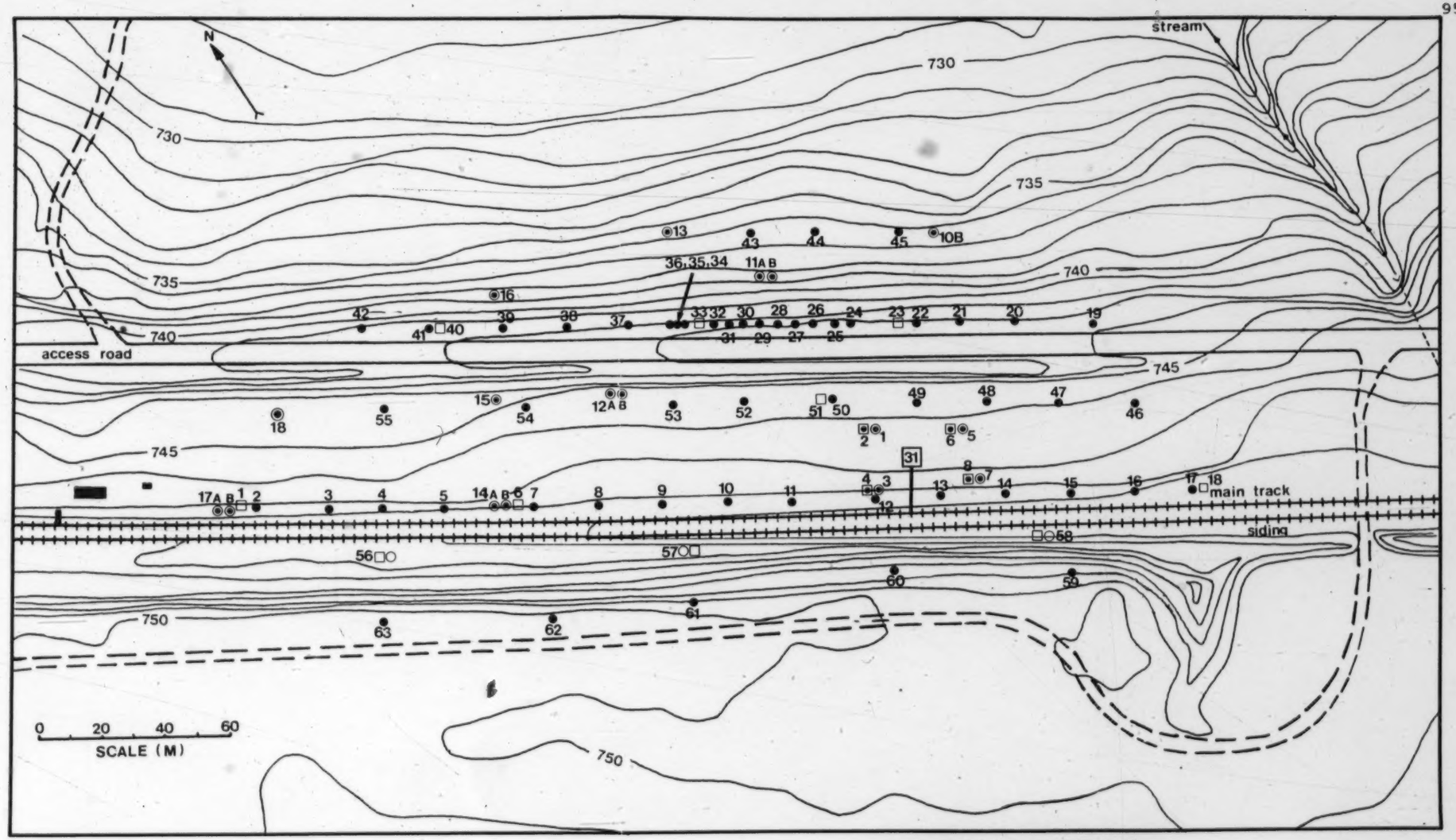

Figure 15. Instrumentation location plan.

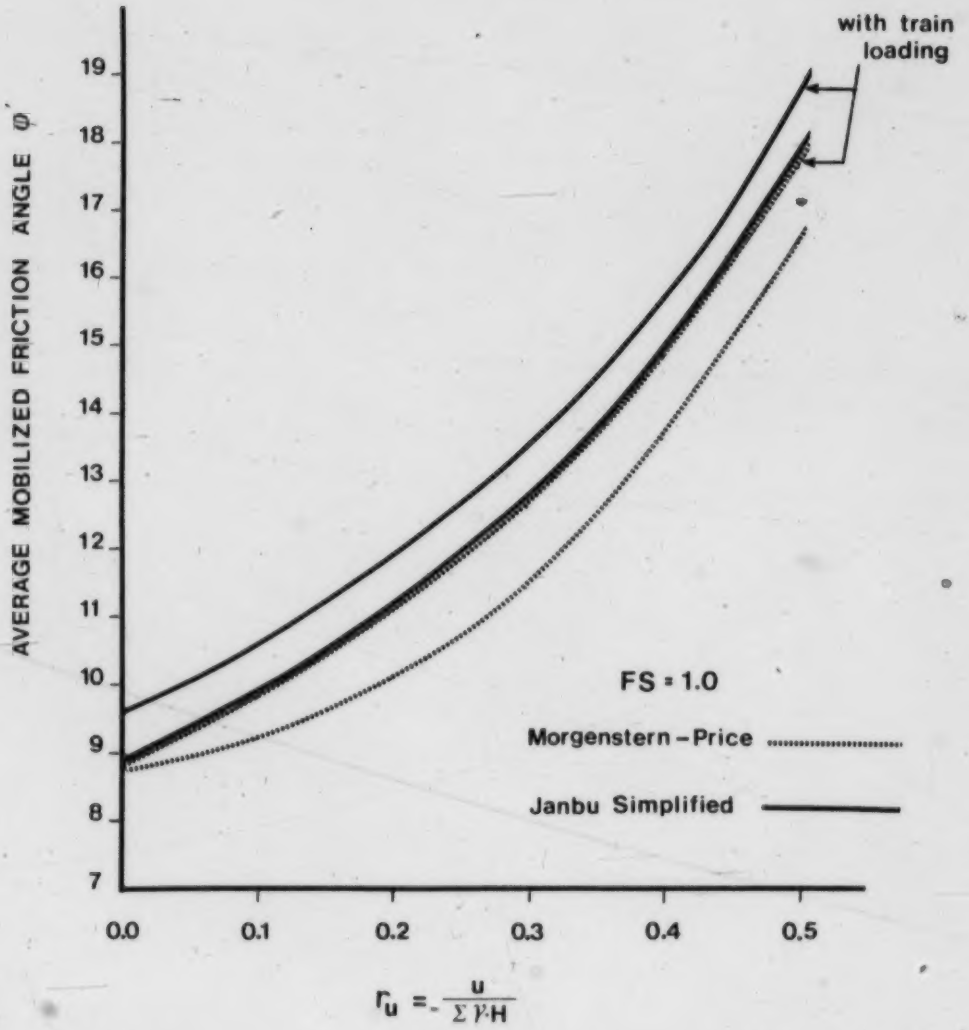

Figure 16. Parametric back-analysis of 1983 Slide using Morgenstern-Price and Janbu Simplified analyses.

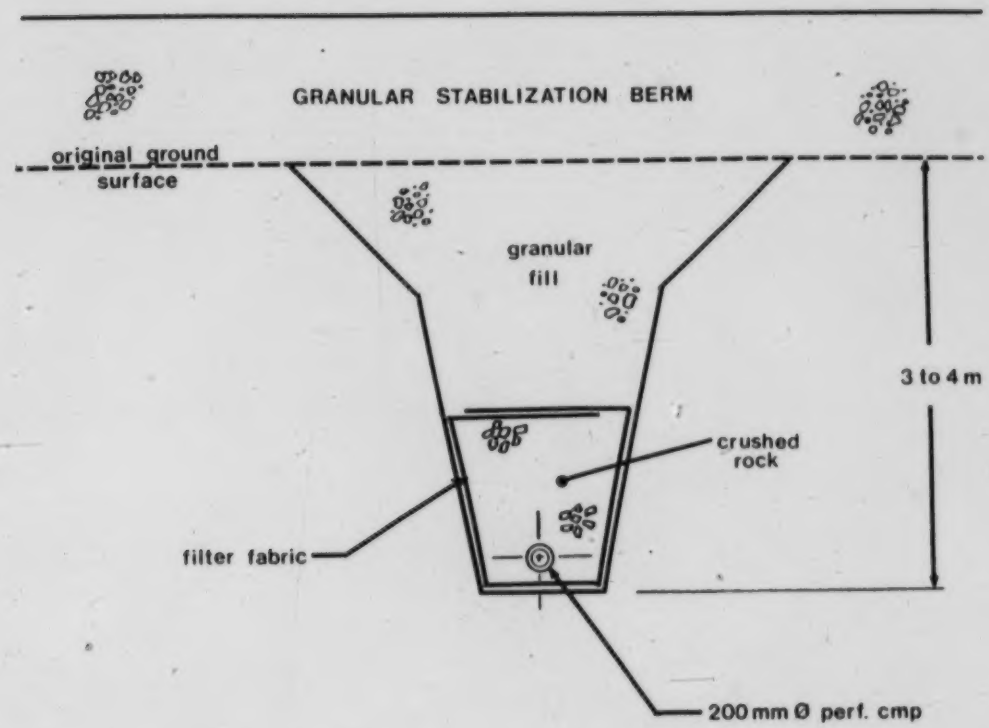

Figure 17. Schematic cross-section of counterfort drainage trenches.

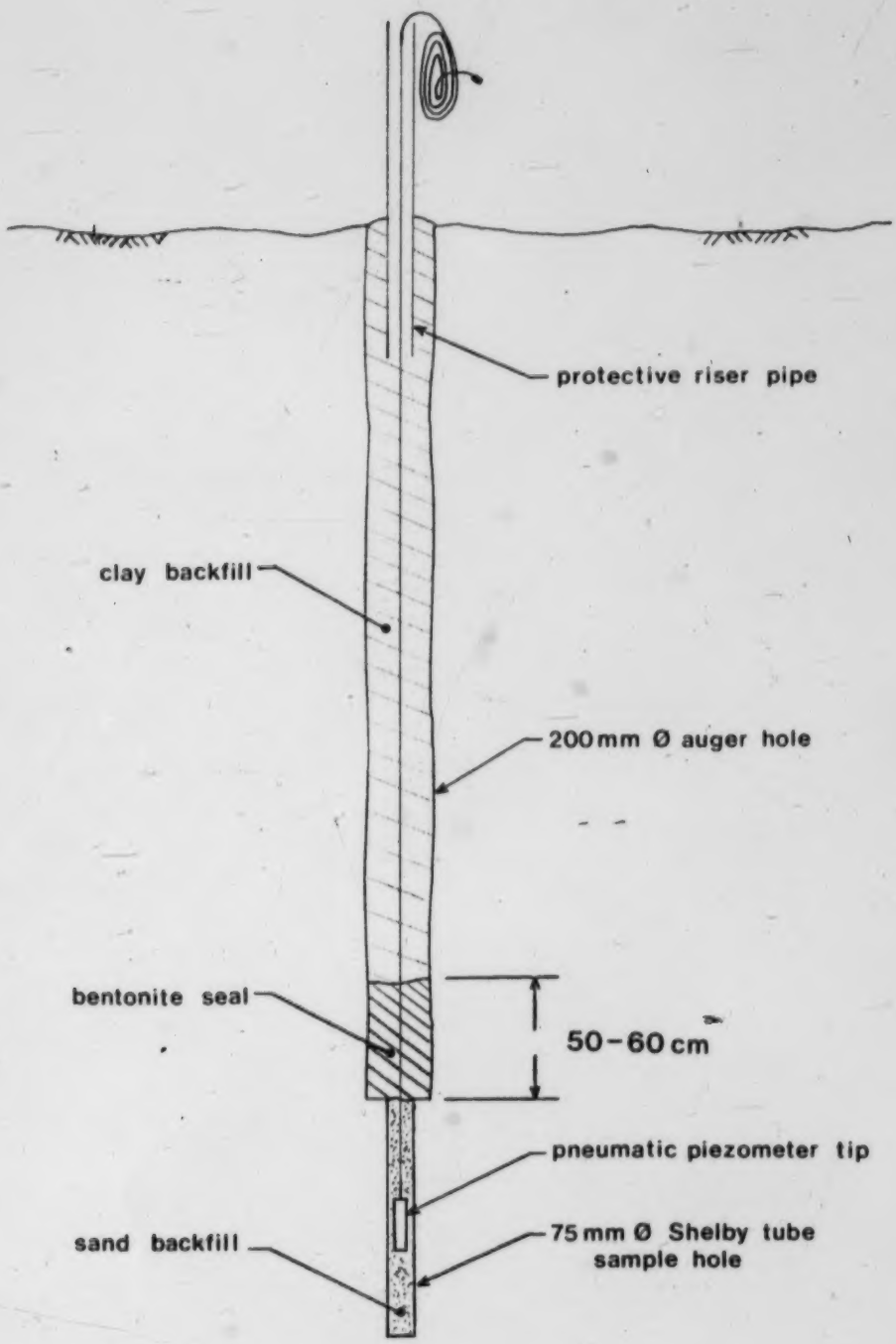

Figure 18. Schematic cross-section of typical pneumatic piezometer installation.

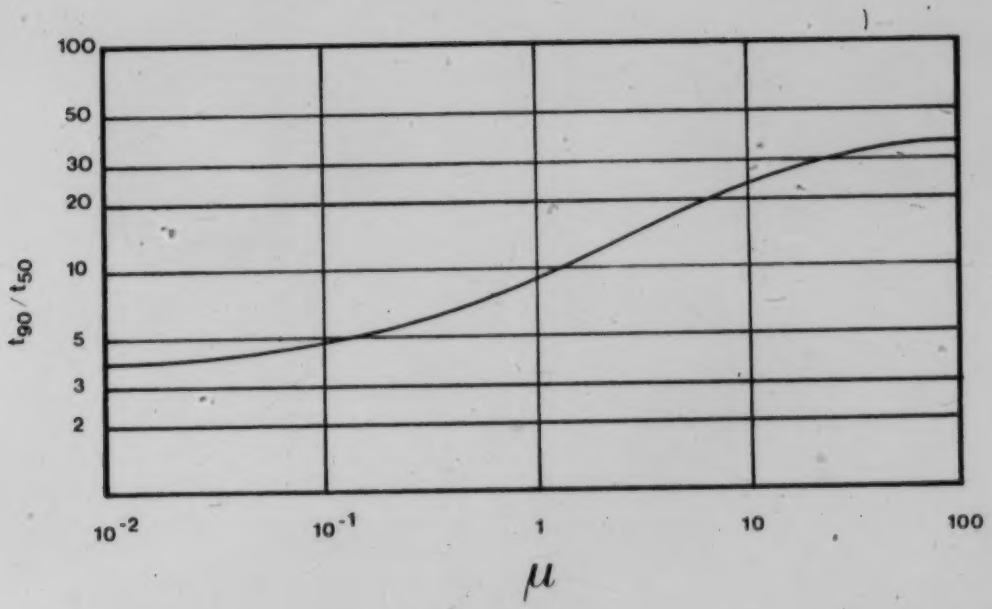

Figure 19. Relationship between system flexibility and the ratio of t_{90}/t_{50} (from Premchitt and Brand, 1981).

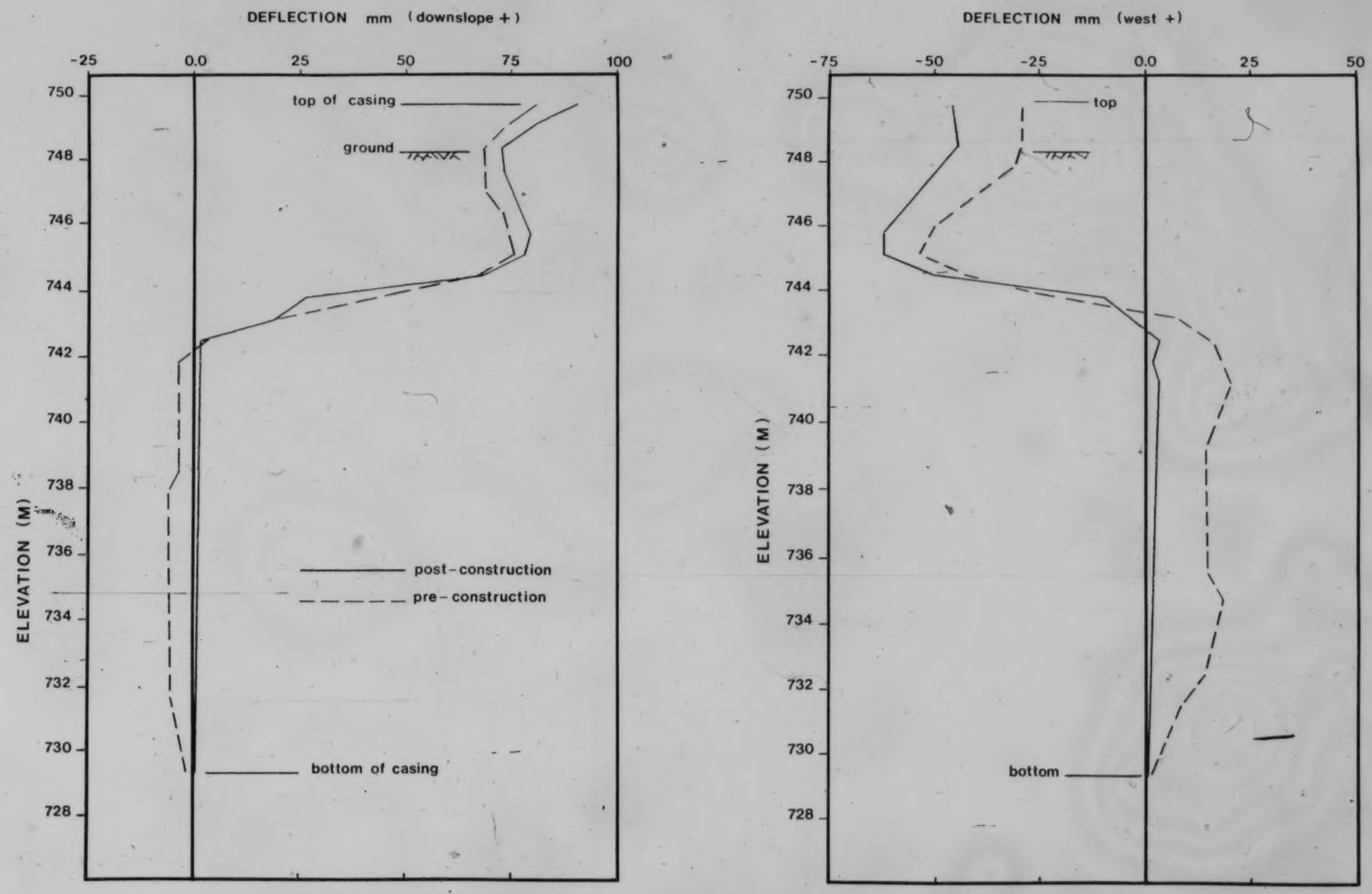

Figure 20. Slope indicator profile record for installation SI84-4.

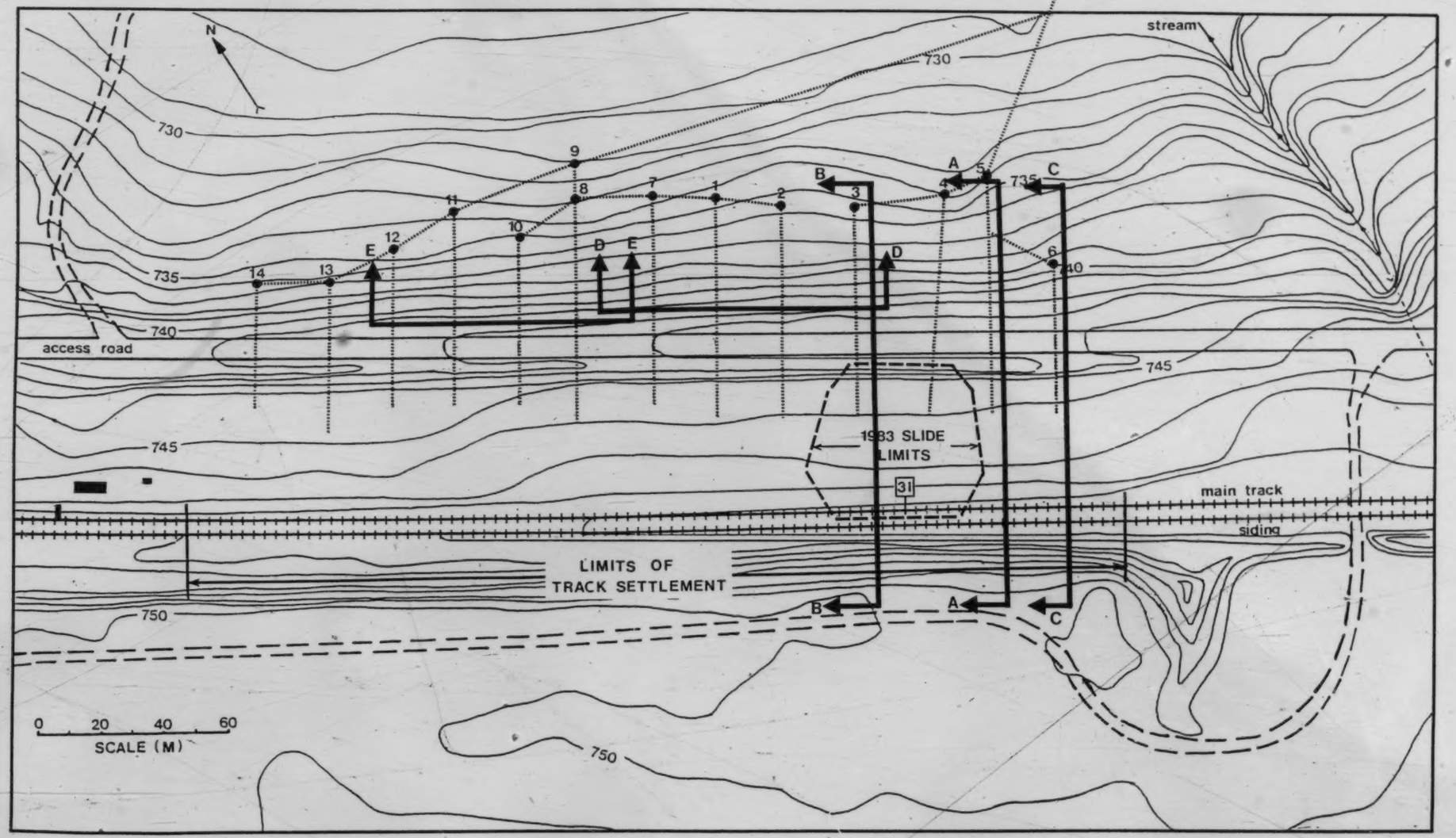

Figure 21. Location of slope cross-sections used in analysis of the influence of counterfort drain installation and stabilizing berm construction.

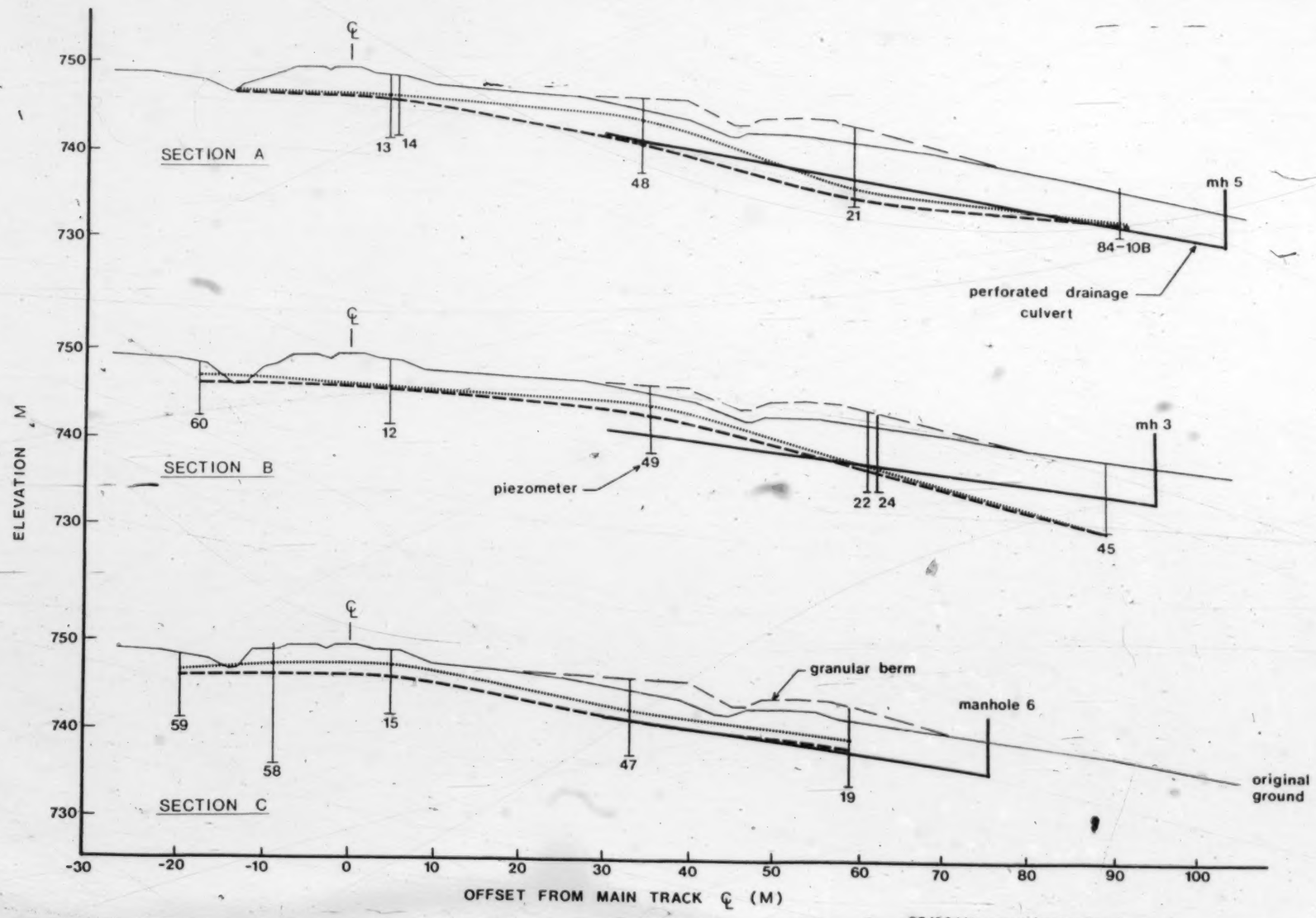

Figure 22. Slope cross-sections taken along the dip of the slope illustrating the piezometric levels present before and after drain and berm construction.

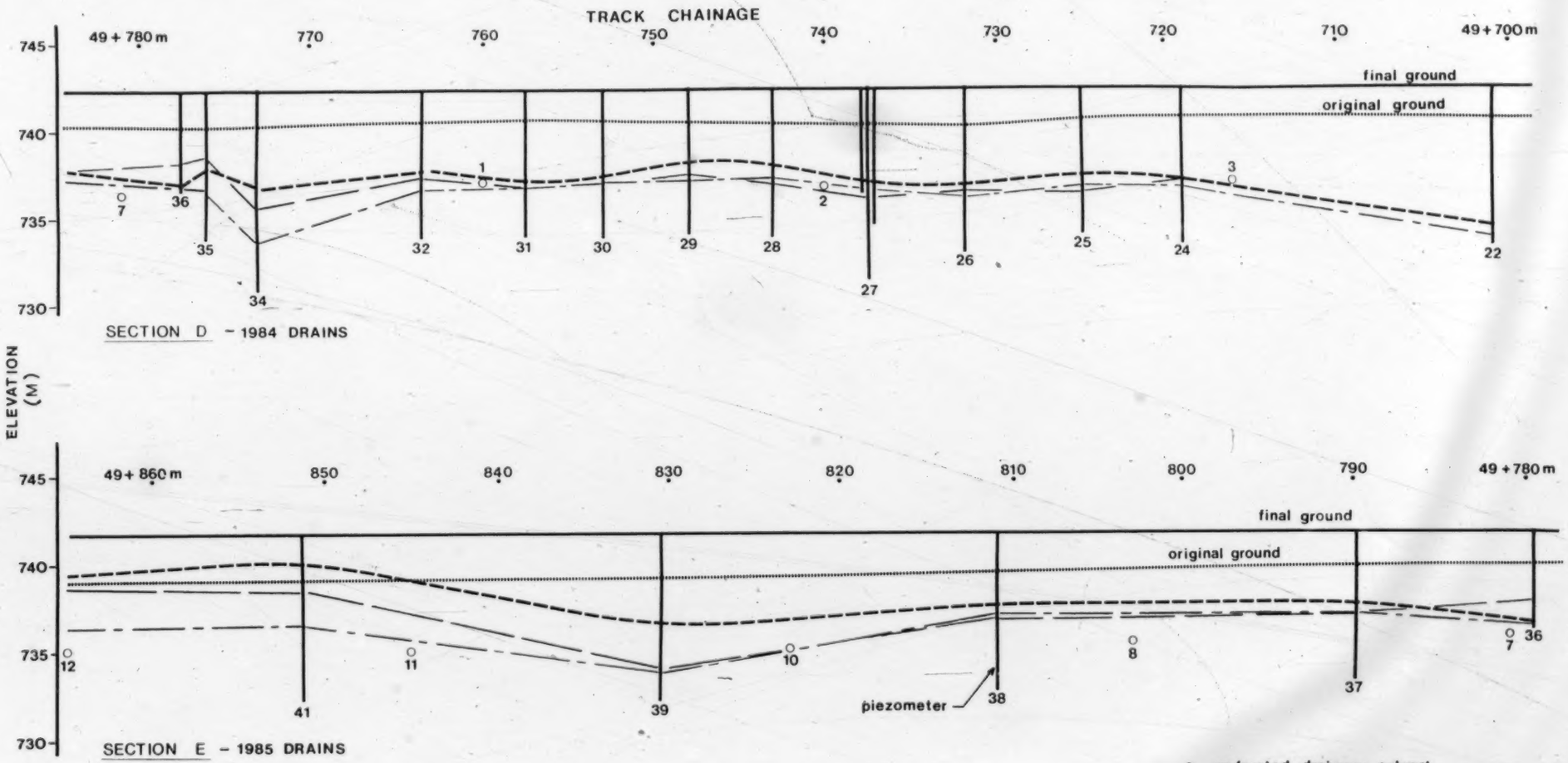

Figure 23. Slope cross-sections taken along strike of slope illustrating drawdown influence of both the 1984 and the 1985 counterfort drains.

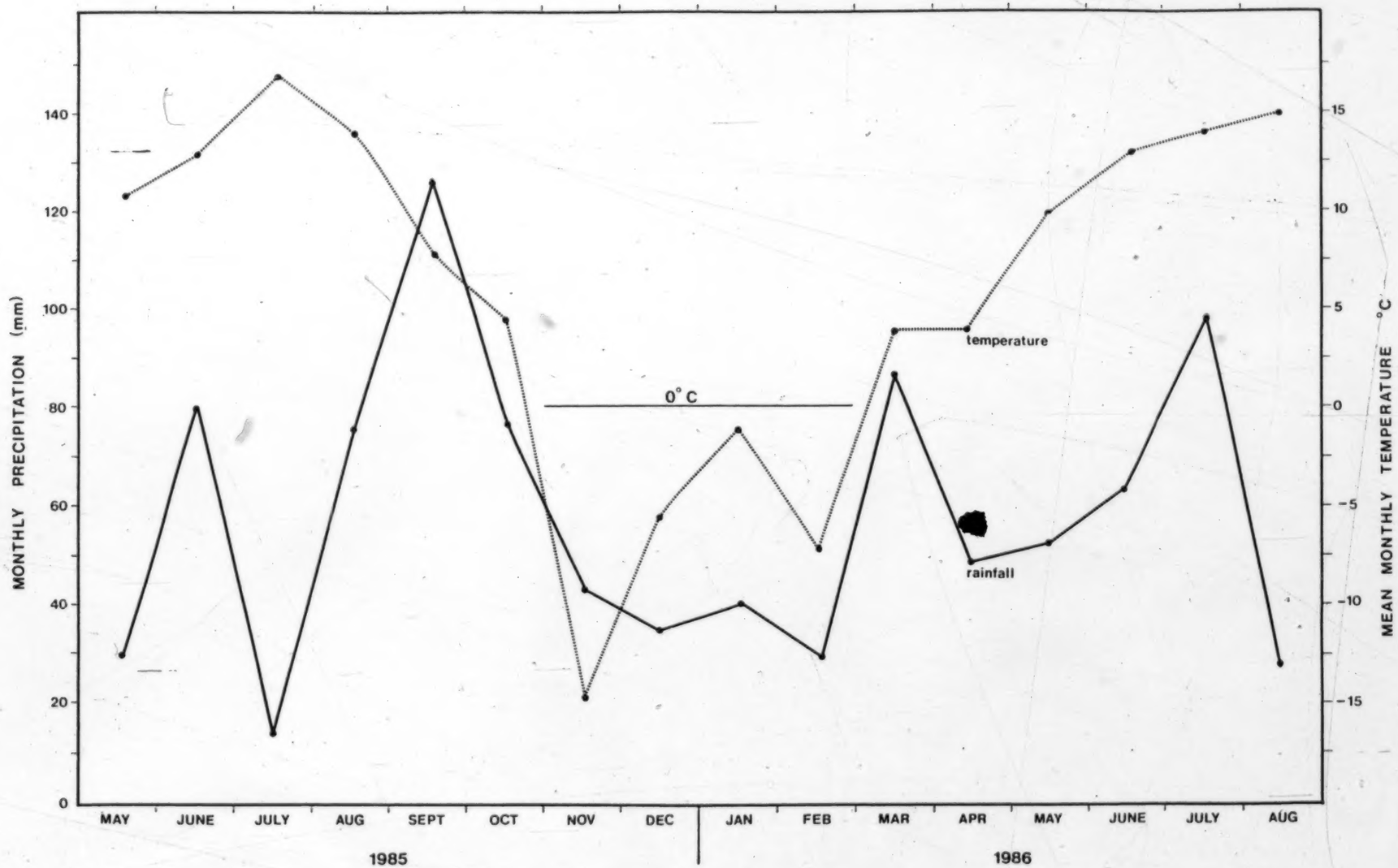

Figure 24. Monthly precipitation and mean monthly temperature data for the period of observation (May 1985 to August 1986).

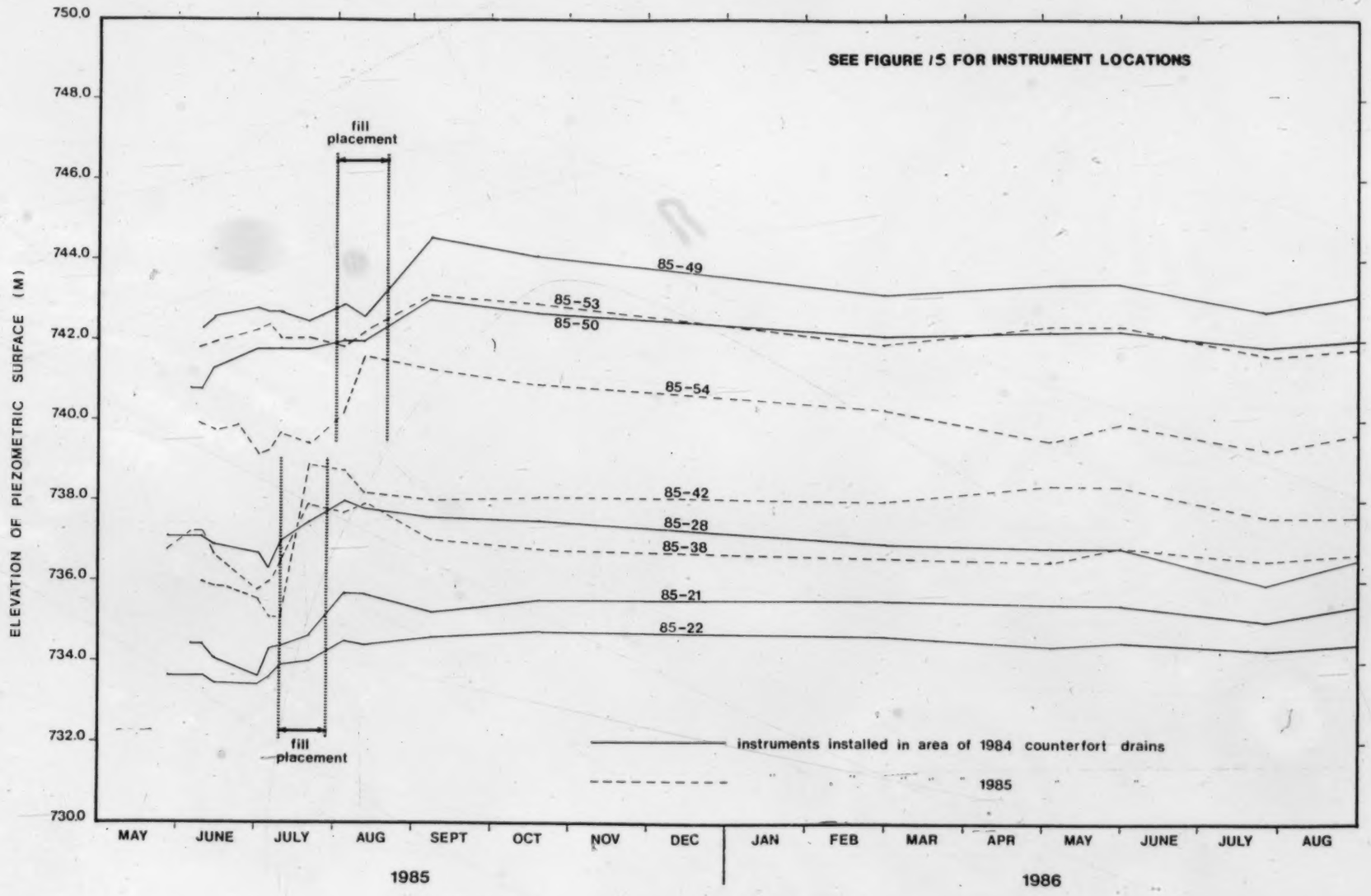

Figure 25a. Piezometric level vs. time plots for period of observation (May 1985 to August 1986).

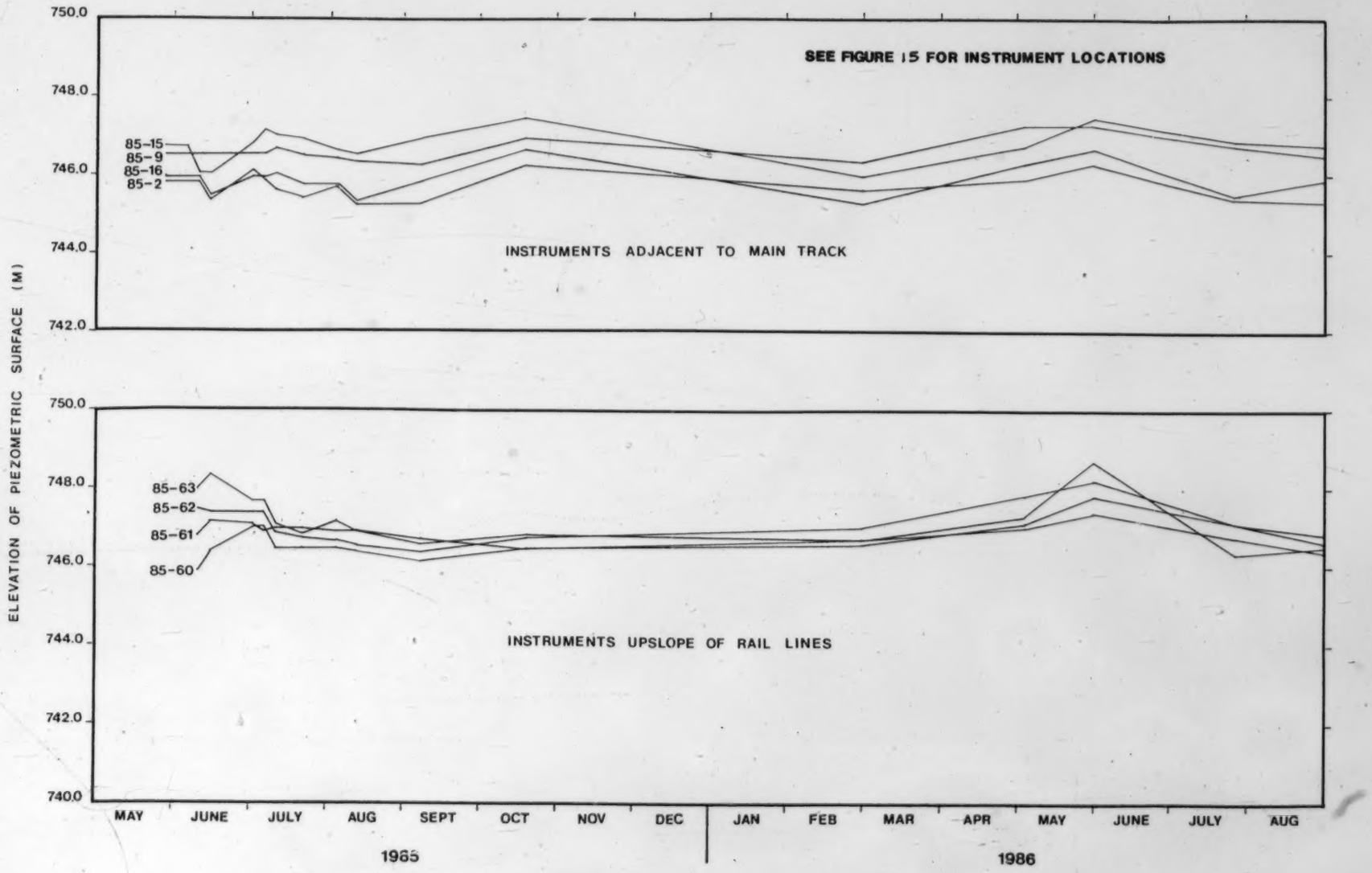

Figure 25b. Piezometric level vs. time plots for period of observation (May 1985 to August 1986).

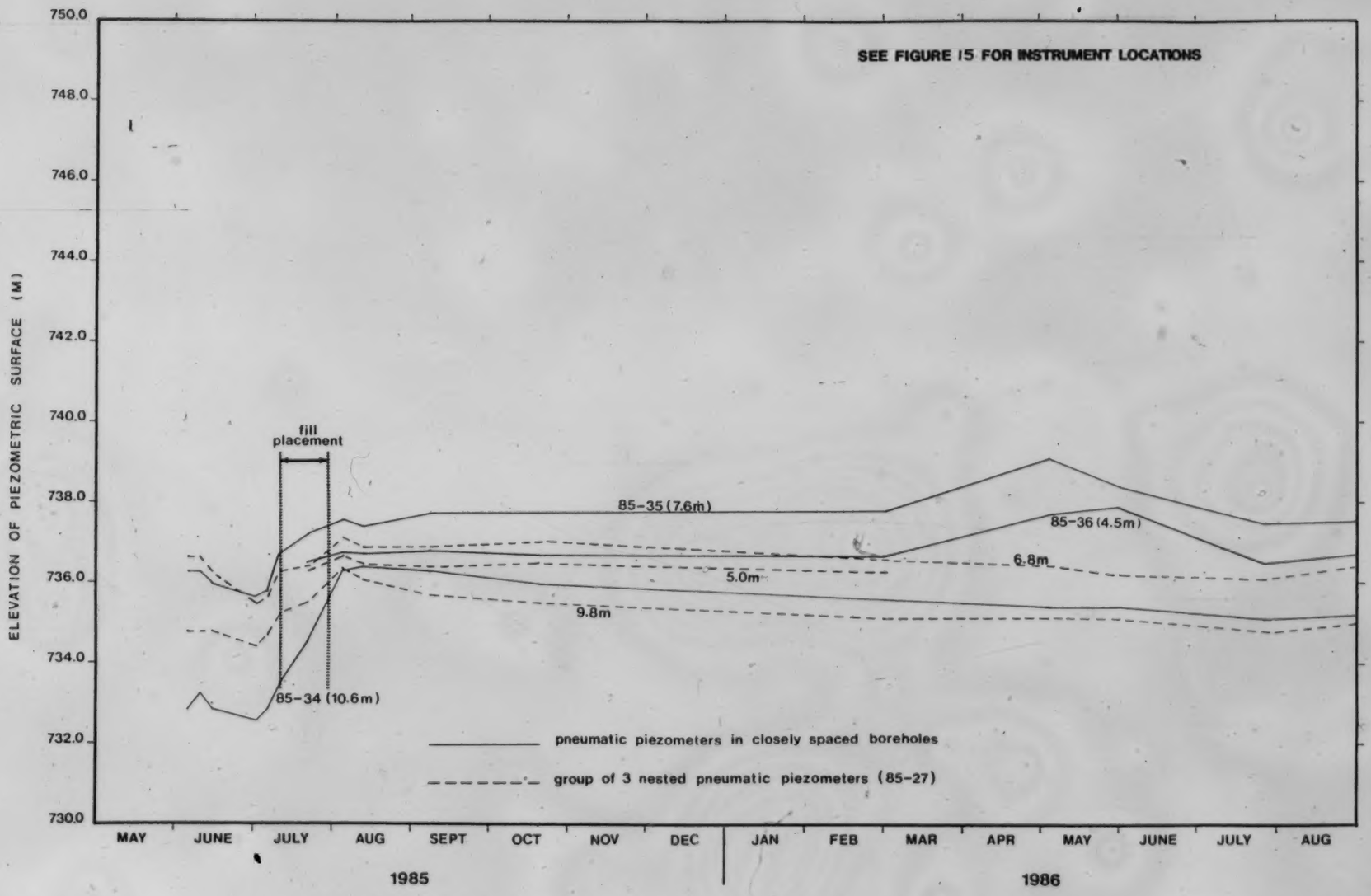

Figure 25c. Piezometric level vs. time plots for period of observation (May 1985 to August 1986).

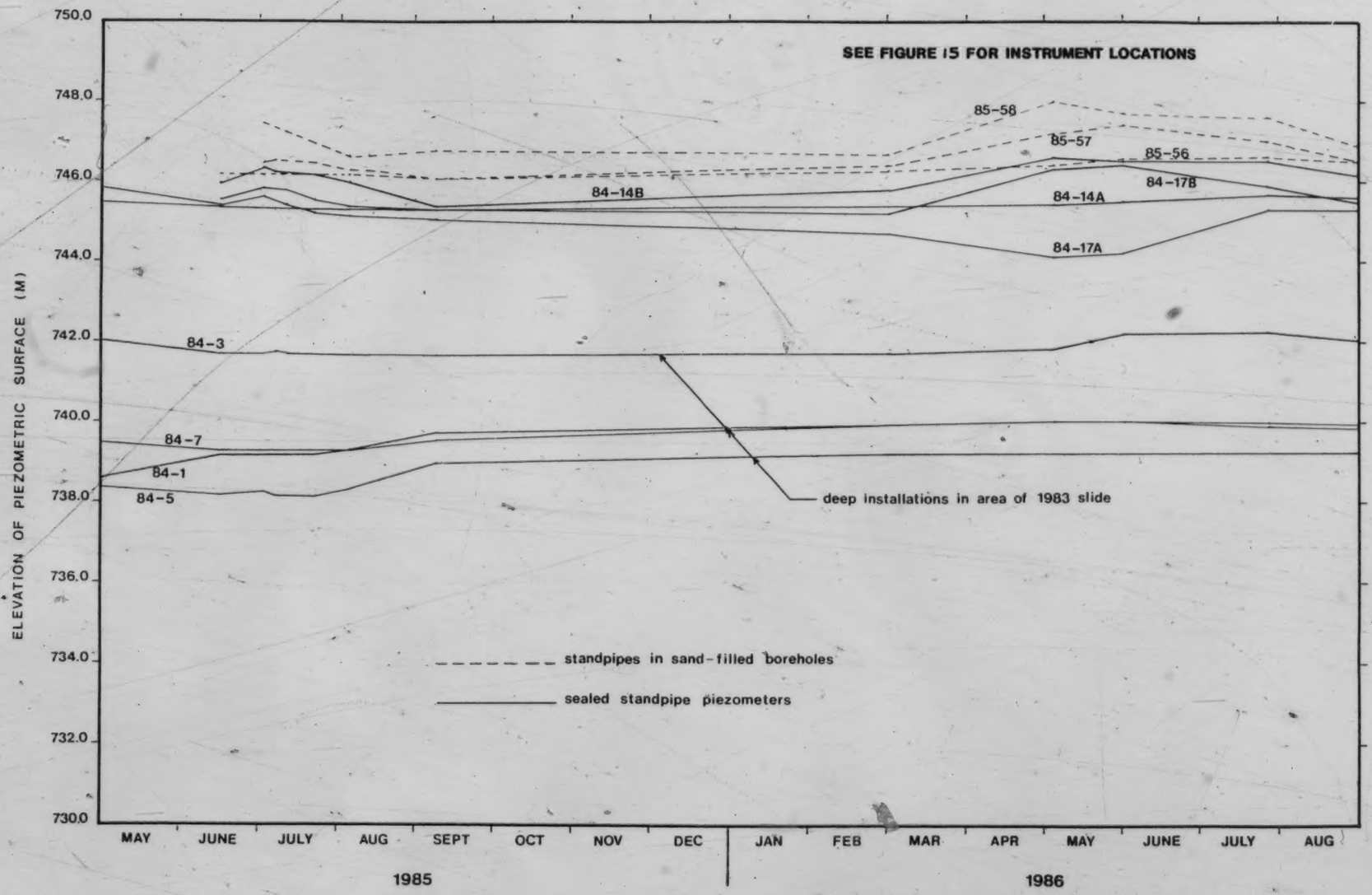

Figure 25d. Piezometric level vs. time plots for period of observation (May 1985 to August 1986).

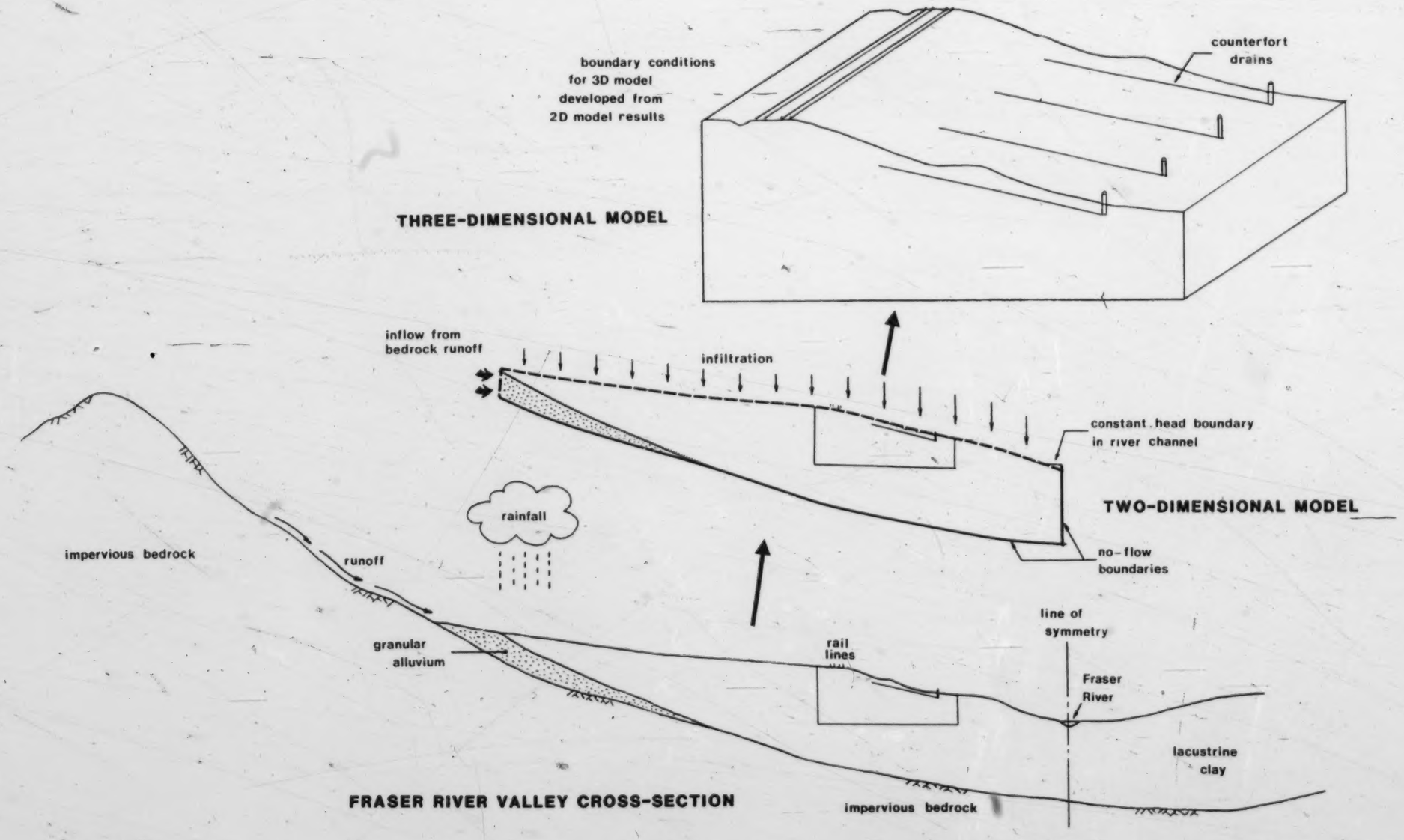

Figure 26. Schematic illustration of groundwater flow model slope geometry for use in numerical modelling

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APPENDIX

THE QUALITY OF THIS MICROFICHE IS HEAVILY DEPENDENT UPON THE QUALITY OF THE THESIS SUBMITTED FOR MICROFILMING.

UNFORTUNATELY THE COLOURED ILLUSTRATIONS OF THIS THESIS CAN ONLY YIELD DIFFERENT TONES OF GREY.

LA QUALITE DE CETTE MICROFICHE DEPEND GRANDEMENT DE LA QUALITE DE LA THESE SOUMISE AU MICROFILMAGE.

MALHEUREUSEMENT, LES DIFFERENTES ILLUSTRATIONS EN COULEURS DE CETTE THESE NE PEUVENT DONNER QUE DES TEINTES DE GRIS.

Plate 1a. Small rotational slides along the roadways near the Mile 30.8 site resulting from seasonally high ground water levels and overcutting of slopes.

Plate 1b. Small rotational slides along the roadways near the Mile 30.8 site resulting from seasonally high ground water levels and overcutting of slopes.

Plate 2. View of the Mile 30.8 site looking northwest along the telephone line. Note the offset in the telephone poles indicating a downslope displacement in the area of track settlement.

Plate 3. View of the drainage ditch on the upslope side of the access roadway prior to slope stabilization works. Slope movements resulted in the formation of a toe bulge which can be seen closing off the drainage ditch.

Plate 4. CN Rail hollow-stem auger drilling rig used in the site investigation and instrumentation programs.

Plate 5. View of a counterfort drainage trench during construction.

Plate 6a. View of the site looking northwest along the centerline of the access road during construction. The road grade was increased an average of 2 meters to form a stabilizing berm. Counterfort drains underly the roadway.

Plate 6b. View of the site looking southeast during the berm construction. The rail line appears top right and the manholes at the end of the drainage culverts appear top left.